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Durability of Building Materials & Components 8

Service Life and Asset Management

VOLUME 2

Durability of Building Assemblies and Methods of Service Life Prediction

edited by
MICHAEL A. LACASSE
and
DANA J. VANIER

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Dr. Michael A. Lacasse is a materials scientist and building science engineer who since 1991 has worked as a Research Officer at the Institute for Research in Construction, of the National Research Council Canada. His work focuses on the development of methods to assess the long-term performance of building materials and components and the durability of building envelope assemblies. Since joining the IRC, he has been active in various CIB and RILEM technical committees related to performance, durability and service-life prediction. He currently co-ordinates the activities of the joint CIB/RILEM technical committee W080/175-SLM in which issues related to the prediction of service life of building materials and components are reviewed.

Dr. Dana J. Vanier's current activities relate to service life, asset management and product modeling. He is the project manager of IRC's Building Envelope Life Cycle Asset Management (BELCAM) project, that is researching tools and techniques to help asset managers predict the remaining service life of roofing components and to better manage their assets. Dana is also involved in IRC's Municipal Infrastructure Investment Planning (MIIP) project, whose goal is to assist asset managers predict the life cycle inspection, maintenance, repair and replacement costs of urban infrastructure capital assets. He is an editor of ITCon, the *Electronic Journal of Information Technology in Construction*, and a Board Member of CIB W078, Information Technology in Construction.

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Edited by

Michael A. Lacasse and Dana J. Vanier
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Preface

Asset managers are faced with many difficult decisions regarding when and how to maintain and repair their constructed facilities. The reasons behind these difficult decisions are the growing fiscal constraints, an increasing maintenance deficit and a substantial repair backlog. In addition, asset managers' resources are being challenged from all sides: they are also being asked to cut cost, privatise operations, outsource responsibilities, reduce maintenance and increase efficiency. These problems are exacerbated by the lack of: (1) usable data, (2) information and knowledge related to maintenance, (3) repair and service life prediction techniques, and (4) tools to assist the asset manager to make proper inspection, maintenance, repair and replacement choices.

Many of these problems would be solved if more information exchange were present in the construction industry regarding service life and asset management. Many organisations have in recent years recognised the scope of the problems facing asset managers. Unfortunately existing opportunities offer only partial solutions to many of the problems in asset management. Hence there is a pressing need for research and development in this field. For example: (1) what maintenance data are required to predict service life of materials and components, (2) what are the life cycle costs of maintenance and repair alternatives, (3) what techniques are available for service life prediction, (4) how can performance be quantified using objective and representative indicators (5) how can the likelihood of failure be calculated and the consequences of failure be identified, and finally, (6) how to integrate this information to allow asset managers to make cost-effective and objective decisions on maintenance, repair and renewal alternatives?

It is within this framework that the conference themes were chosen and to which many authors have provided significant contributions to the advancement of their respective research fields. The collected body of works, presented herein, focuses on topics related to integrating existing knowledge together with the more recent contributions on service life and durability of construction and building materials, components, and assemblies. Particular emphasis has been placed on information technologies in construction and its' role in integrating knowledge through classification schemes, product models, information retrieval systems and collating and dissemination technologies.

The proceedings have been published in four volumes. The first volume, "Service Life and Durability of Materials and Components" contains four parts each reflecting those contributions made to the durability of specific building materials, namely: concrete, masonry, wood, and polymers. Within the part on concrete materials, there are several sub-sections including service life prediction of concrete structures, strength and durability, corrosion, and papers on issues related to design and innovation. The second volume "Durability of Building Assemblies and Methods of Service Life Prediction", contains six parts, the first three of which provide those papers related to environmental characterization, building facades, and roofing materials and components. Within this last area, roofing maintenance management has likewise been included as this provides the collected works in the area of roofing as a distinct part of the proceedings. The final three parts of this volume focus on methods of service life prediction, including probabilistic and reliability methods, generic methods, and lastly, development and use of guides and standards. "Performance, Service Life Prediction and Sustainable Construction" is the third volume in which the collected works are presented in four parts, starting with maintenance management, and followed by life cycle costing and economics, performance and serviceability of buildings (CIB W060 workshop), and life cycle analysis and sustainable, durable design. The final volume "Information Technology in Construction", represents those papers submitted to theCIB W78 workshop of the same name and is prepared in eight sections including computer-integrated construction, the construction process, decision-support systems, design systems, life cycle models, product modeling, process modeling, and virtual reality.

Collectively these contributions provide a selected overview of approaches, methods, techniques, tools, systems and technology used to develop knowledge on the service life durability of construction and building materials, components and assemblies and the integration of this information towards asset management. As such, it is hoped the many whohave contributed to these proceedings, as well as those who in the future will partake in perusing its many pages, will gain in an understanding of the issues, a knowledge of the techniques, some insight into the research, and healthy discernment of useful technologies.

Finally, in editing the various papers and thereafter preparing them for final copy, the editors have made every effort to produce a faithful reflection of the authors' contributions based on their revised manuscripts. However, as might be expected in proceedings of this size, and irrespective of their many efforts, the editors concede that unfortunately, but ultimately, there are likely to be found errors and omissions. These we genuinely regret and would like to express in advance our apologies for any such oversights.

M.A. Lacasse and D.J. Vanier,
Editors, 8DBMC
Ottawa, 1999

Foreword

This conference represents the 8th triennial in a series, in which the previous seven were held in Ottawa (1978), Gaithersburg (1981), Espoo (1984), Singapore (1987), Brighton (1990),Omiya (1993), and Stockholm (1996). The evolution of these collected works over the past two decades attest to the persistent and incessant pursuit of knowledge on the durability of materials, components and assemblies whether in regards to fundamental or applied research, or the most recent technological innovations. These conference proceedings have been representative of the state-of-the-art on service life prediction methods and the durability of constructed facilities.

More recently, attention has been placed on deriving from these sources of information, a means of providing practitioners with ready access to the vast body of knowledge encompassed within studies on the "durability of materials". Moreover, recent advances in information technologies permit its use as an integration tool and dissemination device, and a suitable interface to the current and evolving needs of practitioners. It is in this context that the 8DBMC provides a unique venue for exhibiting the most recent innovations in information technologies as a tool to link various research fields associated with service life prediction.

Consequently, this set of conference proceedings focuses on topics related to integrating existing knowledge together with more recent contributions on service life and durability of construction and building materials, components and assemblies. This includes work in the area of service life prediction methods and modeling, field studies and evaluation methods, degradation mechanisms, material characterization, and other related items. It is also concerned with providing a forum for new knowledge in the area of service life directly related to life cycle economics, maintenance management, performance requirement modeling, probabilistic and reliability methods, as well as, promoting the integration of this knowledge towards practical applications with emphasis on the use of information technologies as an integration tool.

These proceedings will be of interest to construction researchers, practitioners and designers concerned with service life prediction and durability as well as asset mangers interested in obtaining knowledge about the most recent innovations currently being considered as management tools. Information technologists will find considerable scope towards the application side of their emerging technologies.

These volumes are therefore a singularly valuable set of documents and a worthy addition to the previous collected works and to the annals of the Institute for Research in Construction.

Sherif Barakat
Director General
Institute for Research in Construction
National Research Council Canada
Ottawa, 1999

Acknowledgements

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M.A. Lacasse and D.J. Vanier
Editors, 8DBMC
Ottawa, 1999

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PART FIVE

ENVIRONMENTAL CHARACTERISTICS AND MODELING

84 AN AUSTRALIA-WIDE MAP OF CORROSIVITY: A GIS APPROACH

I.S. Cole, G.A. King, G.S. Trinidad, W.Y. Chan, and D.A. Paterson

CSIRO, Construction and Engineering, Melbourne, Australia

Abstract

This paper presents two approaches to modelling atmospheric corrosion across regions of Australia. One is based on a purely statistical analysis of a large database of corrosion rates and climate. The other is based on combining such a statistical approach with modelling the sources and distribution of airborne salinity. The statistical approach is applied to data derived from a corrosivity survey of the State of South Australia. In this survey, corrosion rates were measured over 475 sites scattered over 420,000 km^2. The corrosion data was combined with climatic data and a model developed by regression analysis. This model proved highly accurate at describing the variation in corrosion rate across the state. Unfortunately, when the model was used to estimate corrosion rates across the Australian continent it proved highly inaccurate. As a response, a second model was developed. In this model, the role of the sea as a source of airborne salinity was explicitly included as were the factors (including wind direction) controlling the distribution of airborne salinity across the continent. This model was found to have reasonable accuracy, although additional factors (notably the effect of industrial pollution) still need to be incorporated.

Keywords: Atmospheric corrosion, mild steel, airborne salinity, sea state, whitecaps, wind direction, regression analysis

1 Introduction

Many authors have conducted exposure programs in which climatic parameters are monitored and statistical procedures (usually regression analysis) are used to derive relationships between the measured corrosion rates and the climatic parameters (Atteraas and Haagenrud 1982; Haynie 1982; Benarie and Lipfert 1986; Spence and Haynie 1990). Unfortunately, despite the detailed work carried out in this area and the many models proposed the scientific community

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has accepted no uniform model. In this paper, a model is derived from the combined analysis (multiple regression) of the results of an extensive corrosivity survey of South Australia and climatic data. It will be shown that this model can comprehensively account for the variation observed in the corrosion rate of the exposed plates. However, when the same model was applied to data from the whole of the Australian continent, it failed dramatically. This failure was instructive in a number of ways. Firstly, it underlined the limitations of models derived from "limited" data sets. In fact, the South Australian data set was the most extensive and reliable ever derived in Australia and in its scope exceeded that of the major European and American programs. However, the model contained intrinsic assumptions regarding a number of key characteristics of the climatic (including wind) patterns and pollutant patterns which, although valid for South Australia, were not valid for the rest of the continent. The key point is that a statistical model can only incorporate sources of variation that are relevant for the data set analysed. The converse is that climatic factors that can cause variations in zones outside the data set, if constant in the analysed data, will not occur in the models.

Given these limitations of statistically derived models, an attempt was made to develop a model more firmly based on understanding the processes controlling corrosion. In fact, the processes controlling corrosion across a continent are complex and it proved impossible to develop a pure process-based model. Thus, this paper presents a composite model where, although the generalised formulation may be based on an understanding of processes, many of the details of the model are derived from empirical methods or from fitting procedures (such as regression). The development of this model has been very informative in defining factors that may control the extent of corrosion across Australia.

2 A model of South Australian corrosivity

An initial corrosivity model for the State of South Australia was derived from statistical analysis.

2.1 The South Australian data set

South Australia is a southern/central sate of Australia, located between the latitudes of 38° and 26° south and the longitudes of 141° and 129°. The south-east of the State is moderately populated and has a temperate climate, while the majority of the rest of the state is semi-arid or arid. From 1991 to 1993, a corrosivity study was conducted over the whole state (420,000 km^2) using 475 sites. The site density varied considerably with the vast majority of sites in the coastal regions. At each site, duplicates of a low-alloy copper-bearing mild steel (BISRA) were exposed on brackets fixed to low voltage electric power supply poles at a height of 3.7 m above the ground and facing north. Corrosion rates over periods of one or two years were determined, however only the two-year data (in micron/year) is analysed in developing the models. Full details of procedures are given by King (1993).

2.2 The South Australian model

The model is based on regression analysis of measured corrosion rate as a function of a range of climatic and pollutant parameters. However, during the corrosivity survey, climatic data was not measured at the sites and thus a data set must be constructed. Site climatic data can be derived from data from meteorological stations using a triangular irregular network, or TIN, model (following the procedures of Okabe et al. (1992)). In addition to meteorological data, data on solar radiation was also derived following these procedures. The climatic and pollutant parameters included in the regression modelling are: solar radiation, average 3 pm RH (RH_{3pm}), average 9 am RH (RH_{9am}), average daily maximum temperature ($Temp_{max}$), average daily minimum temperature, mean temperature, rainfall (mm), and raindays.

In addition, each point is assigned an airborne salinity value. This salinity value is estimated using a simple model. This model makes the following assumptions:

- The salinity at the coast depends on the type of coast.
- A short-range (SRS) and a long-range (LRS) term can model the decrease in airborne salinity with increasing distance from the coast.
- Both of these terms show an inverse exponential decrease with distance from the coast.
- The rate at which the salinity falls off depends on the type of coast.

Formally the salinity factor (Sal) is defined as:

$$\text{Sal} = [\text{LRS} + (\text{SRS} \times s)]/(s + 1) \qquad (1)$$

where s is an integer defined by the coast type, and the units of salinity are mg/m^2.day. The values of s for different definitions of coast type, along with E, are given in Table 1. E in Eqs (2) and (3) is a measure of the salinity at the coast.

Table 1: Parameters for salinity at different coast types

Coast type	s	E
Very sheltered bay	1	5
Partially closed bay	2	15
Sheltered surf or very open bay	3	45
Open surf	4	130

LRS and SRS are then defined as:

$$\text{LRS} = E \times \exp[-\text{dist} \times (s + 1.5)/275] \qquad (2)$$

$$\text{SRS} = E \times \exp(-\text{dist}/\sqrt{s}) \qquad (3)$$

In these equations, dist is the minimum distance from the point to the nearest coast in an "allowable" direction. An allowable direction is one in which wind is observed to originate for at least 5% of the year.

These equations formalise the assumptions listed above. The separation of coast into different types and the assumption that there are long-range and short-range salt distribution terms is based on both measurements of airborne salinity (Cole and Ganther 1996) and on studies of salt production and wave mechanics. The values used for the initial airborne salinity is again based on actual measurements, however the exact formulation of rate of decrease and form of the decrease has been derived as a result of optimising the fit of the experimental data points.

When the salinity factor is included with the climate data, multiple regression yields the following approximations:

$$\mathrm{Ln(Corrosion)} = \mathrm{A} \times \mathrm{Ln}(\mathrm{RH}_{9am}) + \mathrm{B} \times \mathrm{Ln}(\mathrm{Temp}_{max}) + \mathrm{C} \times \mathrm{Ln(Sal)} \qquad (4)$$

where A, B and C are constants derived from the regression analysis.

The mean average error of the model fit with the data is 0.143, which when converted from log gives an error of 15%. All the variables are significant at the 99% confidence level; the accuracy of the model (using Eq. (4)) is represented graphically in Figure. 1 which is a misfit diagram for South Australia. This diagram presents the predicted/measured corrosion rates. It is apparent that 80% of the predictions are within 20% of the measured data, and 97% of the predictions within 40% of the measured data. The above model assumes no effect of terrain. Variables were introduced into the regression analysis to test the sensitivity of the data to terrain factors. Variables included are elevation at site, highest elevation between the site and nearest coast, and the ratio of the last two variables. None of these variables increased the fit of the model in a significant way.

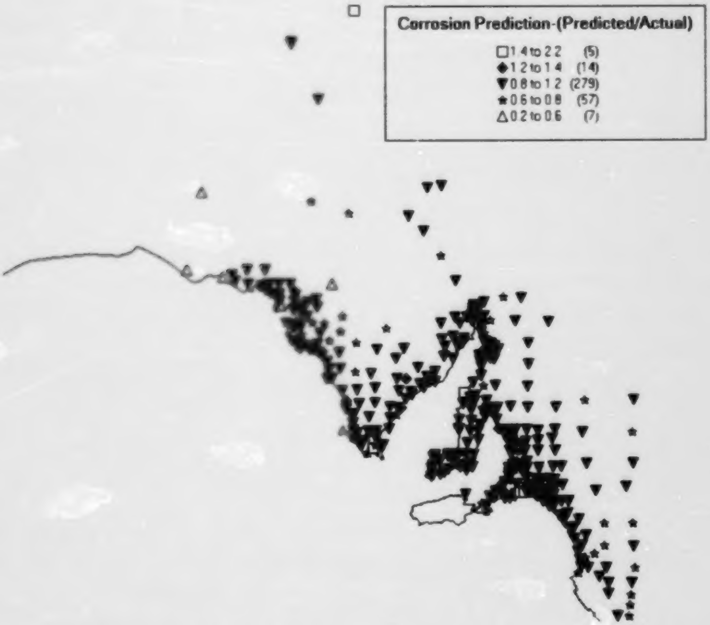

Fig. 1: Misfit diagram for South Australia

3 Application of the South Australian model to the continent

A data set of the corrosion rate of copper-containing mild steel was derived from a variety of published and unpublished data (Egan 1971; Martin 1973; Martin and King 1981; Smith et al. 1994). This data set contained 75 measurements in all Australian climatic zones. The South Australian model was applied to the Australian data set. A misfit diagram is given in Figure 2. The fit is extremely poor with over 41% of predictions being a factor of 2 greater than measured. Further, the model is extremely unreliable along the north-west coast of Australia.

4 Assumptions within the South Australian model

South Australia lies just above the latitude known as the "roaring forties" which is subject to strong westerly winds. The Great Australian Bight lies south of South Australia and is contiguous with the Southern Ocean. The bight is subject to high seas and the exposed coast of South Australia to significant surf. Further, the wind patterns across South Australia are relatively constant, with dominant westerlies shifting to south-westerlies and, in inland regions, to north-westerlies. Thus, for South Australia it is reasonable to assume that for a given beach type, airborne salinity is constant (that is the ocean state around the coast is constant) and that detailed modelling of wind direction is not required. These assumptions are invalid when applied to the Australian continent, as sea activity generally decreases further north while the weather patterns on the west and east coast of Australia are very variable and seasonal.

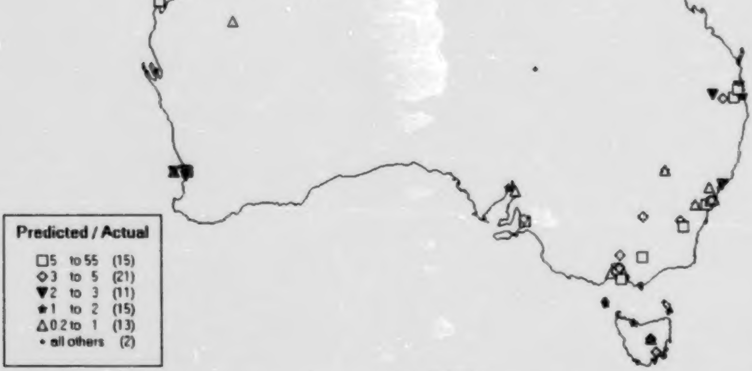

Fig. 2: Misfit diagram for South Australian model applied to Australia

5 The Australian model

The failure of the South Australian model when applied to Australia necessitated the development of a new Australian model. In this model, the effect of salinity sources and the effect of wind on salinity distribution are included explicitly in the model.

5.1 Basic formulation

The model maintains the partition of pollutant and climate effects, and is of the form:

$$\text{Corr} = A \times (\text{Sal})^B \times C \times (\text{Climate})^D \qquad \textbf{(5)}$$

The climate factor may be replaced by time of wetness.

5.2 Effect Of climate

Two climate models are proposed:

$$\text{Climate} = E \times \text{RH}_{3\text{pm}}{}^F \times \text{Raindays}^G \qquad \textbf{(6)}$$

$$\text{Climate} = H \times (\text{TOW-ISO})^I \qquad \textbf{(7)}$$

where TOW-ISO is time of wetness defined by the ISO Standard 9223, i.e. the surface is wet if RH is >80% and temperature is >0°C. This condition is applied to three-hourly data on RH and temperature derived from ten years of meteorological data following the procedures outlined in the South Australian model.

5.3 Salinity

5.3.1 Sources of salinity

Studies of wave activity and airborne salinity over oceans indicate that relationships exist between sea state and salt production on the one hand, and sea state and latitude on the other. Salt aerosols may be generated either by the ocean or from surf, through bursting of bubbles generated by ocean whitecaps, particles torn from the crests of ocean whitecaps by the winds, and breakers on the shore.

An estimate of the sodium (and thus chloride) concentration in the air above the surf zone (Na_{surf}) can be given by (McKay et al. 1994):

$$\text{Na}_{\text{surf}} = e^{(0.23U - 16.72)} \qquad \textbf{(8)}$$

where U is the wind speed at 2 m above sea level (ms^{-1}).

Monahan (1980) demonstrated that the production of aerosol by the bursting of bubbles generated by whitecaps (dF_o/dr) is given by:

$$dF_o/dr = W\tau^{-1}\, dE/dr \qquad \textbf{(9)}$$

where r is aerosol radius, W is instantaneous fraction of the sea surface covered by whitecaps, τ is the time constant characterising the exponential whitecap decay (measured in seconds), and dE/dr is the differential whitecap aerosol productivity, i.e. the number of droplets per increment droplet radius produced by the decay of a unit area of whitecap.

However, Monahan et al. (1982) found that Eq. (9) could be approximated by:

$$dF_0 = 1.373U^{3.41}\, r^{-3} \times 10^{1.19e^{-B^2}} \qquad \textbf{(10)}$$

where B = (0.380 – logr)/0.650.

Monahan et al. (1986) found that aerosol is also produced from spume drops that form as the result of mechanical disruption of wave crests by the wind. This mechanism becomes particularly effective at higher wind speeds and can be approximated by:

$$dF_1/dr = 6.45 \times 10^{-4}\, e^{2.08v}\, r^{-3}\, e^{-D^2} \qquad \textbf{(11)}$$

where D = 2.18 (1.88 – logr).

Total aerosol production is then given by:

$$DF/dr = dF_0/dr + dF_1/dr \qquad \textbf{(12)}$$

To derive salt production, it is also necessary to know the relationship between whitecap coverage and air speed which, according to Monahan et al. (1982), is given by:

$$W = 3.84 \times 10^{-6}\, U^{3.41} \qquad \textbf{(13)}$$

The monthly distribution of whitecaps around the globe has been derived from satellite observation (Monahan et al. 1982). From such data, the average whitecap coverage in the ocean adjacent to the coast has been determined. Using Eqs (10)–(13), the salt production at any point of the coast due to ocean activity can be approximated. From Eq. (9) the salt production from breaking surf can be estimated.

The salinity concentration at the coast is dependent not only on salinity sources, but also the distribution of salinity across the atmosphere and the volume of air which contains this salinity. It is a reasonable assumption that salinity concentration above the cloud level is negligible and thus salt may distribute from sea level to cloud level. The concentration of salinity thus depends on both transport properties of air and cloud levels. At present we have not been able to incorporate these factors into our model. In order to estimate airborne salinity in the model, it is assumed that each coast type has a maximum airborne salinity (according to Table 1) and that these values are then modified by Eqs (9)–(13).

5.4 Transport of aerosols

Aerosol particles produced by breaking surf tend to be quite coarse (10–50 μm) and are concentrated just above the surf. Particles generated from ocean waves are finer and cover a wider range of sizes (0.5–5 μm). Coarse aerosol particles associated with surf tend to deposit close to the coast, while the finer particles from ocean waves may be carried higher into the atmosphere and deposited further from the coast. In fact, aerosol salt can be distributed up to the cloud layer and may be carried very significant distances, with final deposition be a result of scavenging by rain drops (i.e. salt is deposited by rain). These processes are complex and are being studied by the current authors using CFD and other techniques.

The refined Australian model assumes that the deposition of airborne salinity can be modelled by three terms, i.e. a short- (SRS), a medium- (MRS) and long-range (LRS) term.

The incorporation of salinity sources and distribution mode into the salinity formulation leads to the following definitions:

$$\mathrm{Sal} = [\mathrm{MRS} + \mathrm{LRS} + (\mathrm{SRS} \times s)]/(s + 1) \tag{14}$$

$$\mathrm{SRS} = \mathrm{PS(W)} \times E \times e^{(-\mathrm{dist}/\sqrt{s})} \times \mathrm{CalSRS} \tag{15}$$

$$\mathrm{MRS} = \mathrm{PO} \times [(E - G) \times e^{[-\mathrm{dist} \times (s + 1.5)/275]}] \tag{16}$$

$$\mathrm{LRS} = G \times e^{(-\mathrm{dist}/H)} \tag{17}$$

where PS(W) is the surf salinity production term derived from Eq. (8), CalSRS is a procedure that equates SRS to zero for coastline adjacent to very sheltered bays, and PO is a term to take into account the variation in salinity produced from the ocean, and is derived from Eqs (10)–(13). The values of G is determined by data fitting to be 5, while H is derived from a combination of data fitting and reference to existing Australian data. Keywood et al. (1997) proposed that the distribution of salinity in rain as a function of distance could be modelled by a sum of two inverse exponential terms (short and long range), with the long range having a decay constant of 600 (in km).

5.5 Effect of winds

In the model outlined in Eqs (14)–(17), "dist" is to a given point on the coast from the point for which the corrosion rate is being determined (the modelled point). The Australian coast is represented by 5000 points and for any modelled point, a radius of influence around the modelled point is defined. This circle is then broken into wind quarters and the average salinity contribution (AvSal) from the coastal points in each quarter is determined according to Eqs (14)–(17). The weighted salinity factor is then given by:

$$\text{Weighted salinity} = \sum \mathrm{AvSal} \times \text{Wind freq} \tag{18}$$

The wind frequency used is that of the modelled point derived from meteorological observations at 3 m elevation. This method permits a consideration of the effect of wind distribution on salinity.

5.6 Accuracy of the Australian model

When the new model is applied to the Australia-wide data, it shows a significant improvement over the South Australian model. In Figure 3 a misfit diagram is presented, and the majority of points (64%) fall between 0.6 and 1.2. The greatest number (19%) of outliers fall between 0.2 and 0.6. Of these, 10% are associated with industrial, 5% with severe marine and 4% with other locations. The model does not as yet account for effects of industrial pollutants, and thus the high misfit is to be expected. Overall the mean average error of the model is 36% and thus the model is reasonably accurate at predicting corrosion across a continent. Further, the errors in the model are now informative and highlight fields in which more work is required.

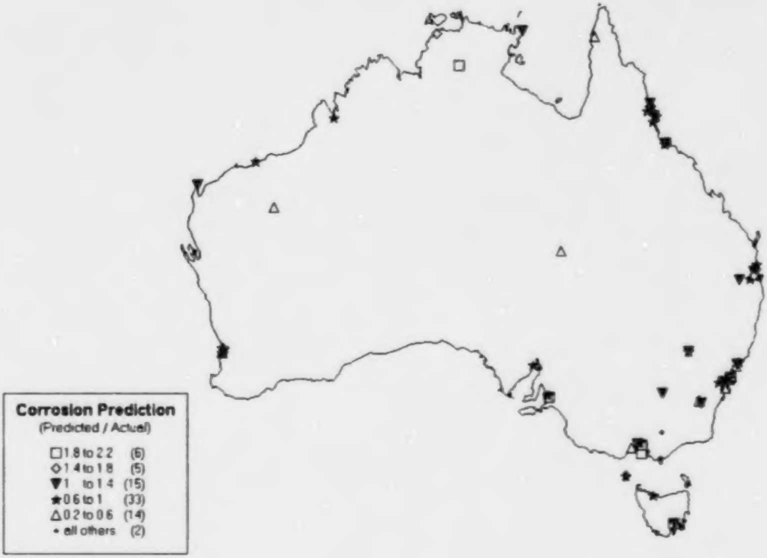

Fig. 3: Misfit diagram for Australian model applied to Australia

6 Conclusions

Two approaches to developing atmospheric corrosion models for regions in Australia have been outlined in this paper. The first is based on regression analysis of large databases of corrosion and climate data, and the second is based on combining such regression analysis with a more detailed modelling of the processes controlling salinity sources and distribution. The following conclusions can be drawn.

- It was found that the regression approach produced models that proved an accurate description of the variation in the data of the databases from which they were constructed. However, these models could not account for the variation observed in a data set derived from a wider range of climates.
- In accounting for corrosion rate across the Australian continent, the role of sea state in controlling salinity production should be considered.
- The use of a salinity distribution term that contains short-, medium- and long-range factors gives rise to satisfactory estimations of corrosion rate.

7 References

Atteraas, L. and Haagenrud, S. (1982) Atmospheric corrosion in Norway. In *Atmospheric Corrosion* (ed. W.H. Ailor), John Wiley & Sons, New York, pp. 873–91.

Benarie, M. and Lipfert, F.L. (1986) A general corrosion function in terms of atmospheric pollutant concentration and rain pH. *Atmospheric Environment*, Vol. 20, No. 10, pp. 1947–58.

Cole, I.S. and Ganther, W. (1996) A preliminary investigation into airborne salinity adjacent to and within the envelope of Australian houses. *Construction and Building Materials*, Vol. 10, No. 3, pp. 203–7.

Egan, F.J. (1971) Effects of environmental factors on the corrosion of steel. *Australian Corrosion Engineering*, Vol. 17, pp. 9–16.

Haynie, F.H. (1982) Evaluation of the effects of microclimate differences on corrosion. In *Atmospheric Corrosion* (ed. W.H. Ailor), John Wiley & Sons, New York, pp. 286–308.

Keywood, M.D., Chivas, A.R., Fifield, L.K., Cresswell, R.G. and Ayers, G.P. (1997) The accession of chloride to the Australian continent. *Australian Journal of Soil Research*, Vol. 35, pp. 1177–89.

King, G.A. (1993) Corrosivity mapping – a sensitive and cost-effective means of characterising a region's level of atmospheric corrosion. Proc. Corrosion 93, NACE, New Orleans, USA, 7–12 March 1993, Paper No. 638.

Martin, K.G. (1973) Atmospheric corrosion of metal cladding in Papua New Guinea and Melbourne. *Australian Corrosion Engineering*, Vol. 19, pp. 19–27.

Martin, K.G., and King, G.A. (1981) Corrosivity measurements at some Australian cities. *Corrosion Australasia*, Vol. 6, No. 4, pp. 10–6.

McKay, W.A., Garland, J.A., Livesley, D., Halliwell, C.M. and Walker, M.I. (1994) The characteristics of the shoreline sea spray and the landward transfer of radionuclides discharged to coastal sea water. *Atmospheric Environment*, Vol. 28, pp. 3299–309.

Monahan, E.C. (1980) Positive charge flux from the world oceans resulting from the bursting of white cap bubbles. Proc. VI Int. Conf. on Atmospheric Chemistry, Manchester, UK, pp. 147–50.

Monahan, E.C, Davidson, K.L. and Spiel, D.E. (1982) Whitecap aerosol productivity deduced from simulation tank measurements. *Journal of Geophysical Research*, Vol. 87, pp. 8898–904.

Monahan, E.C., Spiel, D.E. and Davidson, K.L. (1986) A model of marine aerosol generation via whitecaps and wave disruption. In *Oceonic Whitecaps* (eds E.C. Monahan and G. Macnioaill), D. Reidel Publishing Company, pp. 167–74.

Okabe, A., Boots, B. and Suguhara, H. (1992) *Spatial Tessallations: Concepts and Applications of Voronoi Diagrams*. John Wiley & Sons, New York.

Smith, B.S., Duxbury, E.J. and Moore, B.T. (1994) *Atmospheric Corrosivity of Defence Bases in Northern and Eastern Australia*. Technical Report DSTO-GD-0123. Department of Defence, Australia.

Spence, J.W. and Haynie, F.H. (1990) Derivation of a damage function for galvanised steel structures: corrosion kinetics and thermodynamic considerations. In *Corrosion Testing and Evaluation: Silver Anniversary Volume*, ASTM STP 1000, pp. 208–24.

85 MODEL TO TRANSFORM MEASURED CLIMATIC DATA FROM A LOCAL LEVEL TO A MICRO LEVEL: TEMPERATURES

B. Eriksson and K. Westberg

Royal Institute of Technology, Gävle, Sweden

Abstract

A large part of the building stock and infrastructure will sooner or later fall into disrepair and make the economic and the cultural costs significant due to deficient maintenance. To make the maintenance more efficient and reduce the costs there is a need to predict the service life of building components and materials. In order to estimate the service life of a building component or material, the environmental impact as well as the dose-response function must be known. Many dose-response functions are today available and in this paper we focus on the estimation of the environmental impact. In many countries meteorological measurements have been performed for several years and on many locations. In Sweden meteorological data can be collected for, in principle, any time period in the second half of the century and for several different locations. The number of meteorological stations have gradually increased since the first measurements started in the 1860ies and is today around 900. The measured climatic data must be transformed into the actual impact onto different building materials. In order to perform such transformations the surroundings of the building as well as the building itself must be considered. We have studied one of the most important degradation agents for wood, temperature, and used both meteorological temperature data as well as microclimatic temperature data for one house in Sweden, Berggrenska gården in Gävle. We have studied both the difference between two meteorological stations and the difference between a meteorological station and microclimatic measurements on different positions on a facade.

Keywords: local climate, micro-climate, dose-response function, agents, building material, meteorological data, calculations, WETCORR

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1 Introduction

The building structure is one of the most important areas in a society. In the developed countries, the building stock and infrastructure constitute more than 50 per cent of each country's real capital. To pursue sustainable development and make maintenance more efficient and reduce the costs, there is a need to predict the service life of building components and materials. Service life is defined as the "period of time after installation during which all conditions of a building or a building part meet or exceed the performance requirements" (ISO/CD Part 1 1998).

Principles for service life prediction of materials and buildings have and are being established within national and international standards organisations. An international standardised methodology for prediction of service life of building materials, components and buildings is presently being elaborated in ISO/TC59/SC14 (ISO/CD Part 1-2 1998). The standardised methodology encompasses prediction models which rely on knowledge of degradation mechanisms and dose response functions. Dose-response functions describe the relationship between doses of environmental degradation agents and the observed effect of degradation. Data for the degradation environment at the macro and meso level are available, for example mean values of precipitation, temperature, wind velocities and directions. These kinds of data can after appropriate correction, be used for characterisation of the micro environment at a building.

In the EU-project "Wood-Assess" (ENV4-CT95-0110) (Haagenrud 1998), inspections of the outside wood on old wooden buildings were studied and the environmental impacts were measured. One of the objectives in the "Wood-Assess" project was mapping of environmental risks at the meso and micro scale. In the case of wood the most serious damage is the rotting process. At a specific value of moisture content and temperature, the critical condition for the rotting process to start is reached. A well-known formula that estimates the potential of a climate to promote rotting is Scheffer Climate Risk Index (CRI) (Scheffer 1971). The formula is based on data for precipitation and temperature, data which is mostly available on a meso level. A problem with using the CRI in estimating the likely micro climate is the local variation in meteorological parameters for a specific building. In the "Wood-Assess" project one task is to compare the CRI measurements on local scale with the CRI based on data from the nearest weather station.

Our work is to show how one can use data from available meteorological stations and if this data must be transformed when estimating the degradation on a specific facade. The aim is to use the result in a service life prediction of materials and buildings.

The questions we try to answer are:

1. *Is it possible to use data from a meteorological station to describe the degradation environment at local scale?*
2. *Is it possible to use data from a meteorological station to describe the degradation environment at micro scale?*
3. *Is it possible to do these descriptions at any time period?*

In this paper measured temperatures at the SMHIstation (the nearest official meteorological station) and temperatures measured at KTH-BMG (the research centre) are compared. The deviation of the measured temperatures based on their different locations is discussed in connection to the degradation of wood based on these temperatures. The micro climatic temperatures measured at a building in central Gävle, Berggrenska gården, is also used both to compare against temperatures measured at the meteorological station and in the discussion of the degradation environment.

2 Description of the meteorological stations

The SMHI station used in this work is located in the countryside, approximately 6 km north of Gävle. The surroundings of the station is rather free from both houses and vegetation. Approximately 300 m on the north-east to south-east side of the station there are some small houses, an overview of the meteorological station is shown in Figure 1.

Fig. 1: An overview of the official meteorological station situated some 6 km north of Gävle

The KTH-BMG station on the other hand is in the city of Gävle and located on the roof of the research centre close to the Baltic sea and the river Gavleån, shown in Figure 2. The distance to the central Gävle is approximately 1 km, which is west of the research centre.

The investigated building in this work is the house Berggrenska gården which is situated in the central part of Gävle and also close to the river Gavleån. In the project "Wood-Assess" we placed 14 sensors at the facades of Berggrenska gården that measured both temperature and air humidity. In this project we have used temperature data from three of them.

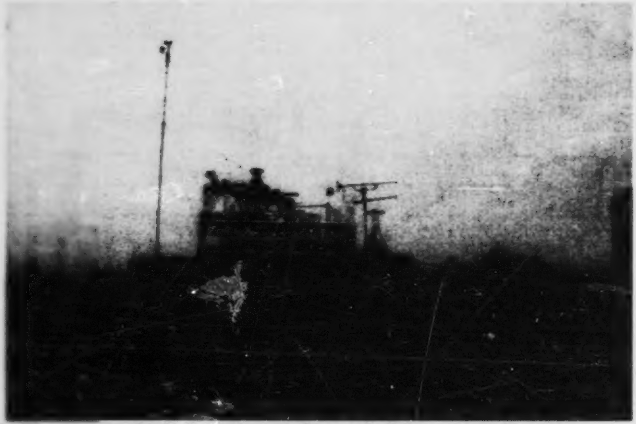

Fig. 2: The meteorological station at the roof of the research centre

3 Difference in environmental impact between the meteorological station and the research centre

The measured data from the meteorological station are given as 3 hour data and the ones measured at the research centre are given per hour. We first have to make the data comparable regarding the time scale, an appropriate time scale is to use 3 hour data. The next step is to compare the measured temperatures between the meteorological station and the research centre.

We have used Equation 1 in the regression analysis of the different temperatures.

$$T_{rc} = m \cdot T_{ms} + b \qquad (1)$$

T_{rc} = temperature measured at research centre, °C
T_{ms} = temperature measured at meteorological station, °C
m = coefficient
b = coefficient
r^2 = coefficient of regression analysis that indicate how well a linear equation given in the analysis shows the relations between variables; a perfect linear correlation gives r^2=1.

The comparison was done in three steps; at first on a yearly basis, second in summer and winter season, and third each month separately. The results from the regression analysis are given in Table 1. The seasonal division is logical to perform since, at least in Scandinavia, the degradation of wood is primarily during the temperate season of the year.

Table 1: Coefficients in the regression analysis for temperature in different time periods under the year 1997

Period	m	b	Temperatures SMHI	Temperatures RC	R^2
Whole year	0.98	1.36	5.9	7.2	0.98
summer	0.95	2.11	11.7	13.2	0.96
winter	0.92	1.07	0.1	1.2	0.97
January	0.83	0.96	-1.8	-0.5	0.95
February	0.96	0.85	-1.2	-0.3	0.99
March	0.95	1.31	1.5	2.8	0.97
April	0.91	1.39	2.7	3.8	0.95
May	0.82	2.77	7.2	8.6	0.92
June	0.79	4.44	13.9	15.4	0.90
July	0.74	6.11	17.0	18.8	0.89
August	0.80	5.26	17.9	19.6	0.91
September	0.90	2.37	11.5	12.8	0.96
October	0.94	1.23	3.6	4.6	0.96
November	0.87	1.08	0.5	1.5	0.98
December	0.87	0.66	-2.1	-1.2	0.97

When studying the regression for the *yearly* data we find that the value of m is very close to one but the value of b implies a temperature difference of approximately 1.4 degrees. The coefficient of r^2 is very close to one which means a good linear correlation between the temperatures.

In the *winter* and *summer* case we have m-values slightly less then one and also r^2-values just below one but the b-values differ and are approximately one for the winter case and two for the summer case.

Next we study the *months* separately. In the first months the m-value is close to one, except for January, but moving into the summer the value gradually decreases until the autumn when is moves back, getting close to one again. For the value of b we see that in the beginning and end of the year the value is around one but during summer it reaches as high as six which means that we are far from measuring the same temperature in that period of time. The r^2-values are between 0.95-0.99 except for the summer months which implies a fairly good linear correlation for the model except perhaps for some of the summer months.

4 Differences in environmental impact between the meteorological station and the investigated building

The next step is to study the difference between the measured temperatures at the meteorological station with the temperatures measured on the facades of the investigated building. The measurements on the facades were performed in the "Wood-Assess" project and we measured temperatures, Time of Wetness at the surface of the wood panels and moisture content inside the wood panels. An example of one of the measuring spots is shown in Figure 3. The microclimatic measurements at the investigated building are given at an hourly basis, so the first we had to do was to transform the data to 3 hours periods in correspondence with data of the meteorological station. We choose to use median values.

Fig. 3. Two of the sensors situated at the north facade of Berggrenska gården

In this paper we are using only the temperature data from sensors on the surface of the facade. First we study these temperatures on a yearly basis, secondly on a seasonal basis and in the end on a monthly basis. These results are given in Table 2.

Table 2: Coefficients in the regression analysis for the three sensors and for different time periods under the year 1997

Period	Sensor 1			Sensor 2			Sensor 3		
	m	b	R^2	m	b	R^2	m	b	R^2
Whole year	1.03	1.42	0.93	1.01	-0.31	0.95	1.04	1.25	0.97
summer	0.99	2.50	0.85	0.96	0.77	0.91	1.00	2.11	0.96
winter	0.87	0.88	0.94	0.88	-0.81	0.88	0.92	0.85	0.94
Jan.	0.73	0.02	0.93	0.61	-1.79	0.85	0.79	0.17	0.94
Feb.	0.91	0.46	0.96	0.92	-1.32	0.91	0.96	0.54	0.96
Mar.	0.85	0.74	0.94	0.94	-1.38	0.89	0.94	0.70	0.94
Apr.	1.01	1.69	0.77	1.12	-0.63	0.85	1.09	1.33	0.91
May	0.90	3.75	0.68	0.96	1.52	0.80	0.99	2.85	0.89
June	0.87	4.96	0.65	0.80	3.42	0.84	0.92	3.79	0.94
July	0.90	5.06	0.56	0.77	4.56	0.77	0.89	4.45	0.92
Aug.	0.87	4.56	0.68	0.77	4.16	0.81	0.88	4.15	0.92
Sep.	0.91	1.99	0.91	0.99	-0.37	0.88	1.01	0.97	0.93
Oct.	0.96	0.87	0.91	1.07	-1.27	0.88	1.03	0.49	0.93
Nov.	0.86	1.36	0.95	0.94	-0.13	0.92	0.94	1.27	0.95
Dec.	0.86	1.29	0.96	0.86	-0.26	0.93	0.93	1.31	0.97

The three sensors where placed on the North facade. Sensor no. 1 on the eastern part of the facade some two meters from the corner, no. 2 in the middle and no. 3 on the western part approximately five meters from the corner.

By using *yearly* data we see that the coefficient *m* is close to one for all three sensors, as is expected, but the values of *b* vary between -0.3 to 1.4 , which means that there is a difference of -0.3 to 1.4 degrees between the meteorological station and the positions on the facade. The value of r^2 is close to one for all three sensors implying a good linear correlation.

Studying the *summer* season the value of *m* is still close to one, but the value of *b* has increased to between 0.8 and 2.5, which means that the difference between the temperatures have increased. The r^2 value is accordingly somewhat lower implying data is more scattered around a straight line.

Next we use *monthly* data and focus on the summer season. We immediately find that for all three sensors the months may-august have very high values of *b*. For sensors 1-2 we also see that the r^2 values are getting much lower and thus less linear correlation using the simple regression formula. For sensor 3 the r^2 value is significantly higher and thus better description of temperatures using the formula. The coefficient *m* is not so far from one except for some month in the middle of the summer.

5 Discussion

5.1 Degradation of wood and meteorological data

In this and earlier work we have focused our studies to the degradation of outdoor wood. To consider only temperatures in these studies is not sufficient. A more detailed analysis of the micro environment in the degradation of wood should also, at least, include precipitation, wind, solar radiation and some biological agents as for example fungi. We are, unfortunately, not able to do a such detailed analysis in this paper. In order to make an estimation model only a few of the degradation agents can be included, normally only precipitation and temperature. We can see that by looking at the formula for Scheffer Climatic Risk Index (CRI) (Scheffer 1971)

$$CRI=\sum[T-2][D-3]/17 \tag{2}$$

where *T* is temperature and *D* is days with precipitation over 0.25 mm and the summation is per month. When the temperature is below 2 °C the *CRI* is negative and is excluded in the summation. Also a dry month with less then three days of precipitation is excluded. In most part of Sweden the months to consider are April-October, which we have called "the summer season" in Tables 1 and 2. We do not consider the degradation of wood in the winter season since the degradation is almost negligible in that period, as is also seen by using Scheffer Climatic Risk Index. For the United States Scheffer, in the 70ies, a recommended how to divide the risk index into three levels for outdoor wood: less than 35, *the least favourable conditions for decay*; 30 to 65, *intermediately favourable conditions*; and greater than 65, *conditions most conducive to decay* (Scheffer 1971).

5.2 Differences between environmental impact at a local level

If we, as in section 3, study the yearly data we can see that the *m*-value is nearly 1.0 and the *b*-value is nearly 1.4. This indicates that the yearly average temperature is 1.4 °C higher at the research centre compared to the meteorological station, and vary nearly equally at both stations. This is reasonable since in the cities the temperatures are normally higher than at the countryside.

If one estimates degradation for a material, that is only depending on the mean value for the year, one can use the data from a meteorological station and add a factor which describes the geographical divergence's. Thus if one wish to estimate the degradation of wood in a building with for example Scheffer Climatic Risk Index, and use data from a meteorological station, one must use a method that take in consideration that the relationship between the temperatures at the stations vary over the year. In our study we can see that the temperature in summer is 2.1 °C higher at research centre than the meteorological station, i.e. 0.7 degrees higher than the yearly difference. As an example we calculate the Scheffer Climatic Risk Index and suppose we have more than 43 days with precipitation over 0.25, in worst case the calculated index will fall in the category *"low risk for decay"* instead of *"high risk for decay"*.

Equation 2 is only applicable when using mean temperatures for each month. In our study we can see that both the *m*-value and the *b*-value vary a lot between the months. The explanation can be that the value of temperature is depending on several agents that are different at different places on a local level and also vary between the different months. The most important of these agents are: *wind (convection), solar radiation, radiation from/to other surfaces, and the humidity of the atmosphere*. We have in some detail, however, studied the differences in wind, both in amplitude and direction, for the meteorological stations. In Figure 4 and 5 we present wind roses, showing the most common wind directions for the different stations. For the research centre, with almost no protection from the Baltic sea, the wind directions during the summer months are mostly from south-west or from north-east. This is not the situation for the SMHI station.

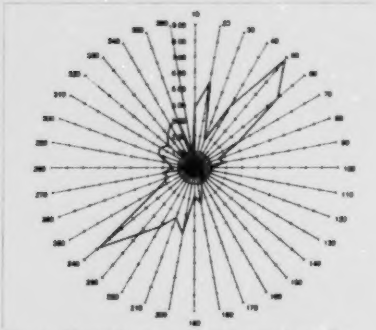

Fig 4: Distribution of the wind at the research centre during the summer of 1997

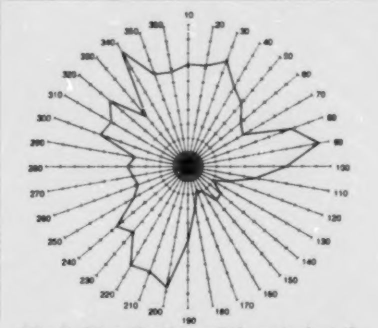

Fig 5: Distribution of the wind at the meteorological station during the summer of 1997

5.3 Differences in environmental impact at micro level

In this work have we chosen three sensors at the North facade, which is directed to the yard in order to reduce the influence of solar radiation and wind (convection). When looking at yearly data the m-value indicate that the temperature vary equally between the meteorological station and the north facade of the investigated building. The b-values indicate that each sensor has also been influenced of something that is unique for the location of each sensor. The most important agents which have an effect on temperature are, besides those we have mentioned in chapter 5.2: *heat transport leaks in the wall and condensation to the surface or evaporation from the surface*. Thus if one is estimating a degradation that only varies with a yearly temperature, one can use the data from a meteorological station and add first a factor that depends on the variation at the local level (see section 5.2) and second add a factor that depends on the micro level. The reason for using two factors are that the first factor is unique for the local area and all buildings there, the second factor is the same for all buildings of the same shape and in the same direction.

When we study the summer season we can see that the m-values indicate that the temperature vary equally at the investigated building and at the station, but b-value indicate that if one is using adding factors, these are different over the year. Further, for both sensor 1 and 2 the r^2 value decreases more then for sensor 3. Sensors 1-2 are located on the lower part of the facade while sensor 3 is on the upper part of the facade. This implies that one must also consider factors that are not controlled in this study. In Figures 4 and 5 we immediately notice that the most common wind directions differ from the meteorological station. That in turn will give different estimations of the precipitation on the facades of, for instance, the investigated building. Using one or the other of the climatic data will give a corresponding difference in the calculated micro environment. In order to overcome such deviations a better model must be used for estimating the microclimate.

The monthly data show us that the r^2-values for some months in the middle of the summer are rather low. By carefully studying the temperatures we found that during these months the temperatures are much higher during some hours in the afternoon, up to 15 degrees. A plausible explanation is radiation from the sun, since the thermocouples were not shielded from the sun. We thought that the buildings in the blocks around the yard should shade the sun, but a close inspection showed that an unfortunate gap between two of the houses. This also implies that the transfer of monthly data over degradation agents from a meteorological station to a spot on a facade must be performed with a function and not only with a factor.

Using equation 2 for estimating the degradation of outdoor wood and considering the value of D as constant (which of course is not the case) we can calculate *CRI* using the meteorological stations and compare with the *CRI* for the actual facade. As an example we can select the month of August. Using the meteorological station as the reference level we will have, with the factor $(D\text{-}3) = 1$ and the mean temperature for August of 17.9°C, a *CRI* of 0.93, using equation 1 and Tables 1 and 2 we get a *CRI* of 1.03 for the research centre and for the three sensors at the north facade of the investigated building we get 1.06, 0.94 respectively 1.05. The results imply that depending on where we measure at the facade and also if we use the meteorological station or the research centre the estimation of the *CRI* will differ between 1-14% and that without even considering the variation in precipitation.

One of the uncertainties with Scheffers Climatic Risk Index, indicated above, is that it only to some degree take into account that rotting of wood requires simultaneously high enough moisture content and temperature for fungi to develop, i.e. the time factor for these events is not recorded. This is taken care of by using the WETCORR technique (Henriksen 1998). The surface temperature and the wetness is two indicators for the risk of rotting. The Scheffer Climatic Risk Index is based on this and so would an index using the WETCORR technique. This can be accomplished, for instance, by summarising the hours when the current is above a certain limit level when the temperature is above a limit for fungus to grow. Using such an index would predict the rotting in much more detail. A difficulty is to measure all building were rotting may occur, since the number is enormous. A better way is to make better models to predict a climatic risk index for wood or actually any material, since Dose-response functions are known for many materials. Using meteorological stations and measurements of the microclimate for different environments would be a starting point in the modelling procedure. For wood we can use the WETCORR technique and make models for temperature and precipitation.

6 Conclusion

This paper gives the following answers to the three questions raised above, in section 1:

a) Data from meteorological stations can probably be used at both local and micro level if they are for the whole year or for summer/winter season. To the value at the meteorological station one should add a factor that describes the difference between a meteorological station and the buildings.
b) For monthly data one must use a more detailed model that also considers several agents that have an influence on the degradation.
c) For shorter time periods a much more complicated model is needed.

To create the factor in a) and the models in b) and c) more research are needed. Even other agents must be examined and also other regions of interest. The project "Wood-Assess" gives more data that can be used in the research.

7 References

International Organisation for Standardisation (1998) *Buildings - Service Life Planning - Part 1: General Principles*, Geneva, ,ISO/CD 15686-1.4.

International Organisation for Standardisation (1998) *Buildings - Service Life Planning - Part 2: Service Life Prediction Principles*, Geneva, ISO/CD 15686-2.3.

Haagenrud S. E., Veit J., Eriksson B. and Henriksen J. F., (1998) EU-project ENV4-CT95-0110 Wood Assess. Project Deliverable no. 5 Final Report. (Wood-Assess doc. Del5/NILU/1998-08-15)

Scheffer T. C. (1971) A climate index for estimating potential for decay in wood structures above ground. *Forest Products Journal vol. 21, no. 10, page 25-31*

Henriksen J. F. and Haagenrud S: E. (1994) The EUREKA project EU615 EUROCARE WETCORR Report from NBS-MK seminar at ABB Conference Centre, Billingstad, 24 November 1993. Kjeller NILU OR 37/94

86 MOISTURE CONTROL IN BUILDINGS — HOW CAN VARYING OUTDOOR CLIMATE BE ALLOWED FOR?

E. Harderup[1] and P.I. Sandberg[2]

[1]Lund University, Lund, and [2]Swedish National Testing and Research Institute, Borås, Sweden

Abstract

This paper describes the general need for secure protection against moisture, and methods for designing the structure in this respect. Ensuring that all the possible moisture loads on different parts of a building have been systematically considered creates the right conditions for designing structures without moisture problems.

An important part of the work of designing structures to deal with moisture loads is to estimate the risk of damage and problems resulting from random variations in the properties of materials, in indoor and outdoor climate and in user/occupant habits. This paper describes, in particular, the effect of a varying outdoor climate and methods for allowing for it when calculating the moisture conditions of various parts of a building and the corresponding reduction in performance of materials and designs,.

By performing calculations for a number of cases over many years (1961-1990) and by studying the spread of the results from one year to another, it is possible to identify the reference (average) year and the most severe (extreme) year. With this information, appropriate safety factors for design calculations can be determined. The paper describes calculated moisture conditions and presents examples of design safety factors for moisture problems encountered in different types of designs.

Keywords: calculation, climate, design, moisture, random, safety

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1 Introduction

An important prerequisite in attempting to achieve sustainable development is the ability to make reliable service life predictions. The gradual breakdown with age, and the corresponding reduction in performance, of materials and designs, are affected by various factors, including the outdoor climate. Normal, but random, variations in the outdoor climate complicate reliable determination of service life, and methods of allowing for the effects of variations in outdoor climate are needed when making service life predictions. This paper describes examples of how these variations can be accommodated when assessing the risk of mildew damage in an exterior wall.

2 Risk and safety philosophy

An important aspect of service life prediction is consideration of safety factors and the overall safety philosophy. Material properties, climate, user habits etc. all vary randomly, and expected effects on materials cannot be expressed as specific values but rather as probabilities of particular types of effects occurring. This probability can then be set against the risk of certain types of damage or adverse effects occurring. This paper describes methods of allowing for the stochastic variations in outdoor climate when making service life predictions. Material properties, user habits and other factors that can vary can then be assumed to be known and constant - although, of course, in a complete analysis they should be considered in the same way as variations in the outdoor climate. There are a number of ways in which allowance can be made for the variations:

- *Determination of the frequency function* for different types of effect, such as temperature, moisture content, UV radiation, ozone concentration etc. This requires the ability to determine the value of the effect on an hour-by-hour basis (or day-byday or month-by-month, as appropriate) over a very long period of time (years), subjected to natural and random variations - in principle, for as long as the life of the building. From this data, the risk of a particular effect or combination of effects occurring can then be easily determined. However, this procedure requires an unreasonably large amount of work: in addition, it involves using climate data for past years as a basis for forecasting that for future years, which means that long-term climate variations cannot be considered.
- *The extreme value method.* See Figure 1A. The effect, illustrated here for moisture conditions, is calculated on the basis of an extremely severe climate, e.g. as the worst that has occurred in 50 years. This value, which can be assumed to present a special risk, is then used for comparison with critical values. This method, too, requires a long series of climate data. However, no additional safety factor is needed, as the necessary safety is already incorporated through choice of the extreme climate situation.
- *The normal value method.* See Figure 1B. Here, the effect is calculated on the basis of normal values of outdoor climate. This data may be those for some particular real year, which is felt to be typical of a number of years, or be a

'synthesised' year, known as a Test Reference Year (TRY). The moisture status value calculated from the normal year values must be corrected in some way, $\Delta\varphi$, in order to represent the value for an extreme year. The advantage of this method is that it is quite easy to obtain or produce data for a normal year, while its drawback is that the necessary corrections for moisture value depend on the type of damage of interest and on the type of climate.

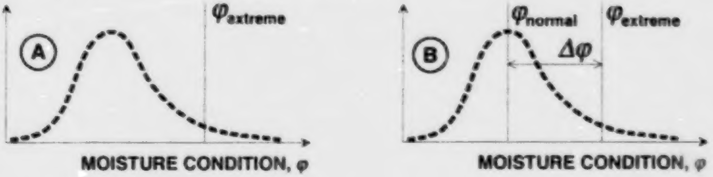

Fig. 1: Frequency curves for moisture condition

A illustrates the extreme value method, in which an extreme moisture condition is calculated from data for an extreme climate. B illustrates the normal value method, in which the value for extreme moisture condition is obtained by applying a suitable correction factor ($\Delta\varphi$) to the normal value.

A research project at the Lund University, Dept of Building Technology, Building Physics, Sweden (Harderup, 1998) is devoted to describing methods of determining the most unfavourable moisture condition that occurs during the lifetime of a building. Suitable correction factors for converting normal year moisture condition values to those for extreme years are being developed by investigating how the moisture condition varies from year to year. By comparing the theoretical results for a long series of years (30 years), it is possible to identify the most unfavourable year. A correction for the variations in climate data can then be determined as the difference between the moisture condition of the most unfavourable year and that of the reference year. This correction can be applied either as an increment or as a factor. The method is illustrated in this paper with an example.

3 Design

The design used in the example is shown in Figure 2, and it is the risk of mildew growth on the outer parts of the wooden studs that is calculated.

The calculations were made using a computer program for calculation of transient temperature and moisture conditions (Harderup, 1998), and considered the case of vapour diffusion in combination with driving rain. The risk was investigated for all compass directions of the facade, in 10° steps from 0° to 350°. The facade was assumed to have a solar absorptance of 0.3 and an emission factor of 0.9. The structure was assumed to be completely airtight, without cracks or gaps, thus ignoring the effect of any air currents. The outdoor climate was in the form of ten sets of data, consisting of hourly values of climate parameters for ten different sites in Sweden. The indoor temperature was assumed to 21 °C, and the indoor relative humidity was calculated on the basis of a moisture input (being the difference between indoor and outdoor vapour concentration) of 0.004 kg/m³.

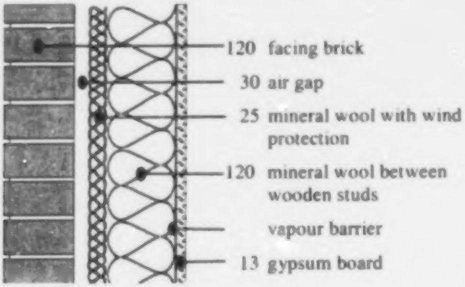
120 facing brick
30 air gap
25 mineral wool with wind protection
120 mineral wool between wooden studs
vapour barrier
13 gypsum board

Fig. 2: A section through the wall in the example

The effects of a ventilated airgap and of the windproof paper have been ignored in the calculations.

The calculations of moisture and temperature conditions were made for a period of 30 years, from 1961 to 1990. The risk of mildew growth was calculated annually, from 16th August of one year to 15th August the next year. The reason for the choice of this period, rather than that of a calendar year, was that the structure normally dries out completely during the summer, with the following moisture absorption during the autumn in combination with drying out in the spring being decisive in determining the risk of mildew formation. The risk is assumed to be dependent on both temperature and relative humidity, as shown in Figure 3.

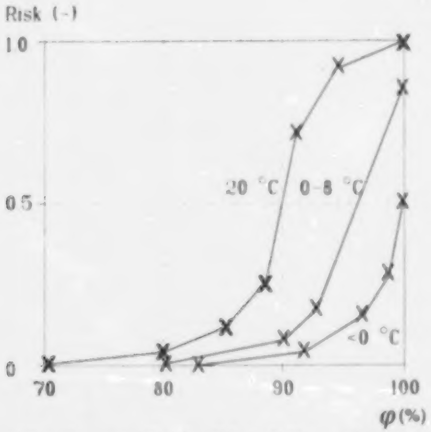

Fig. 3: Risk curve for the growth of mildew (r*) as a function of various moisture conditions and temperatures. From Nevander & Elmarsson, 1991

The risk was calculated for each time increment (Δt), using temperature and RH, and the mean risk, over the period from 16th August to the following 15 August, as follows:

$$r = \frac{\sum r^* \cdot \Delta t}{\sum \Delta t} \qquad (1)$$

4 Results from one site (Bromma)

The results from Bromma are used in order to illustrate some of the principles of evaluation of the theoretical results. This meteorological station is in eastern Sweden, close to an airfield outside Stockholm. The risk of mildew growth, r, has been calculated for 29 twelve-month periods from 1961/62 to 1989/90. Figure 4, showing the theoretical values for a south-facing wall (180°), is an example of the results. The years have been listed in the order of their mildew risk: the mean value is 0.40. The extreme year was 1989/90, for which the risk was 0.45. The year closest to the mean value, the Mean Deviation Year (MDY), was 1961/62.

The correction needed in order to cover the extreme year is therefore 0.45 - 0.40 = 0.05. It can be expressed either as a correction term, $\Delta r = 0.05$, or as a correction factor, $f_r = 0.45/0.40 = 1.125$. Earlier comparisons (Harderup, 1998) have shown that it is a correction term that is most suitable for this type of calculation. The correction term of $\Delta r = 0.05$ in the example above applies only for the 180° direction.

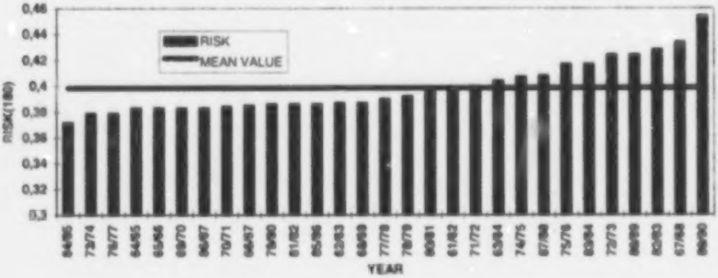

Fig. 4: Yearly mildew risks for 180° facade direction, in order of magnitude. The mean value is 0.40, and the year closest to it is 1961/62

The variation from one direction to another is quite small, which indicates that the effects of solar radiation and driving rain, which are dependent on orientation, are slight. Figure 5 shows the normal values (the mean value of all the years) and the extreme values (the most unfavourable years) of mildew risk for each 30° direction vector.

For practical reasons, it is necessary to determine a value for Δr that can be used for all directions. We could, for example, choose the direction that has the highest value of r which, in 1989/90, was the 150° direction. This direction has a value of $r = 0.46$ and a corresponding value of $\Delta r = 0.06$. MDY is 1961/62. Or we could choose the direction having the highest mean value (for all the years). This turns out to be in the same direction in this case, with Δr still 0.06 and the MDY still 1961/62. We can also calculate the mean value of Δr in all directions, which is found to be just under 0.06, with only very little spread.

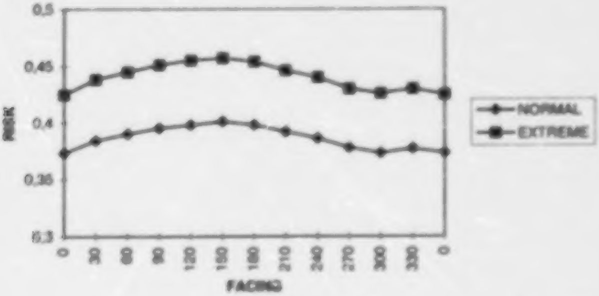

Fig. 5: Normal (mean) and extreme mildew risks of specific direction vectors

It therefore seems as if 0.06 is a suitable value for Δr, but is 1961/62 suitable as the reference year? To investigate this, we have gone through the data for those years that could be considered, and rated each direction so that those years closest to the mean value have been given a score of 0, those next to it a score of 1 and so on. The year with the lowest score, summed over all directions, has then been taken as the normal year: see Table 1.

Table 1: Points for years and directions, in order to identify which year is most representative of a normal year for all directions. The year with the fewest points is taken as the normal year

	Wall facing												
Year	**0**	**30**	**60**	**90**	**120**	**150**	**180**	**210**	**240**	**270**	**300**	**330**	**Sum**
1961/62	3	0	0	0	3	0	0	1	2	2	4	3	18
1962/63	4	7	8	8	5	9	8	8	8	5	5	6	81
1963/64	6	3	3	3	6	4	3	5	5	6	7	5	56
1971/72	5	1	1	1	0	1	1	0	0	0	2	1	13
1977/78	12	8	7	6	12	8	5	4	6	7	12	11	98
1978/79	2	5	5	5	4	3	4	3	4	4	3	4	46
1980/81	1	4	4	3	2	2	2	2	1	3	1	2	27
1981/82	0	2	2	4	1	5	11	6	3	1	0	0	35

The table shows that 1971/72 is the most suitable for selection as the normal year. With one exception (for direction 0°) it is very close to MDY in all directions. 1961/62 could also be used.

The corresponding table for the extreme year (Table 2) shows a very clear ranking of the years, regardless of the facade direction. 1989/90 was the most difficult in all directions, which was probably due to that year having had difficult temperature and humidity conditions, rather than to driving rain, which is direction-dependent.

Table 2: Points for years and directions, in order to identify which year is most representative of an extreme year for all directions. The year with the fewest points is taken as the normal year

	Wall facing												
Year	**0**	**3**	**6**	**9**	**12**	**15**	**18**	**21**	**24**	**27**	**30**	**33**	**Sum**
1967/68	1	1	1	1	1	1	1	1	1	1	1	1	12
1972/73	4	3	3	3	4	3	4	5	4	4	4	4	45
1974/75	6	6	6	6	6	8	8	8	8	6	6	6	80
1982/83	2	2	2	2	2	2	2	2	2	2	2	2	24
1983/84	5	5	5	5	5	6	5	4	5	5	5	5	60
1988/89	3	4	4	4	3	4	3	3	3	3	3	3	40
1989/90	0	0	0	0	0	0	0	0	0	0	0	0	0

The mildew risk for the normal year, (1971/72 - mean value in all directions) is 0,38, while for the 1989/90 extreme year it is 0.44, which gives the same value for Δr, 0.06, as before.

The above therefore says that, for determination of the mildew risk in walls for the Bromma site, 1971/72 should be taken as a normal year, with the result being corrected by a term of $\Delta r = 0.06$ in order to allow for the extreme year conditions.

5 Results for other sites

The same calculations as for the example above have been made for the other ten sites in Sweden. Table 3 shows the results from the three models used in clause 4.

It can be seen from the table that there is no single year that can serve as a common reference year for the whole of Sweden. Different places have different reference years, and the years differ somewhat between themselves depending on the method used to decide on the reference year. If we look more closely at the differences in theoretical risk, r, for the different reference years, we find that they are often small. We can use the points method to find a common reference year, and it tells us that 1978/79 can be used as a reference year for the whole country without difficulty.

The correction factor varies, as shown in Figure 6, being least in the north of the country, where the climate varies least from one year to another as the result of driving rain.

Table 3: Reference year and correction, Δr, for the method with the highest extreme value, the method based on the highest mean value and the method based on assigning points to the individual years

	'Highest extreme value'			'Highest mean value'			'The points method'	
Site	**Direction**	**Ref. year**	**Δr**	**Direction**	**Ref. year**	**Δr**	**Ref. year**	**Δr**
Bromma	300	1961/62	0.06	300	1961/62	0.06	1971/72	0.06
Frösön	200	1963/64	0.03	270	1963/64	0.03	1977/78	0.04
Jönköping	200	1979/80	0.08	130	1961/62	0.08	1964/65	0.11
Karlstad	220	1978/79	0.05	230	1978/79	0.05	1978/79	0.05
Kiruna	200	1985/86	0.02	190	1963/64	0.02	1963/64	0.03
Luleå	250	1965/66	0.01	220	1970/71	0.03	1979/80	0.01
Ronneby	220	1968/69	0.06	230	1968/69	0.06	1980/81	0.07
Sturup	200	1987/88	0.06	230	1987/88	0.06	1986/87	0.04
Säve	200	1986/87	0.06	220	1986/87	0.06	1974/75	0.06
Söderhamn	250	1987/88	0.06	210	1980/81	0.07	1963/64	0.06

For practical use, we can therefore take 1978/79 as the reference year and apply a correction term, Δr, which varies from 0.10 in southern Sweden and on the coast to 0.05 for the rest of the country.

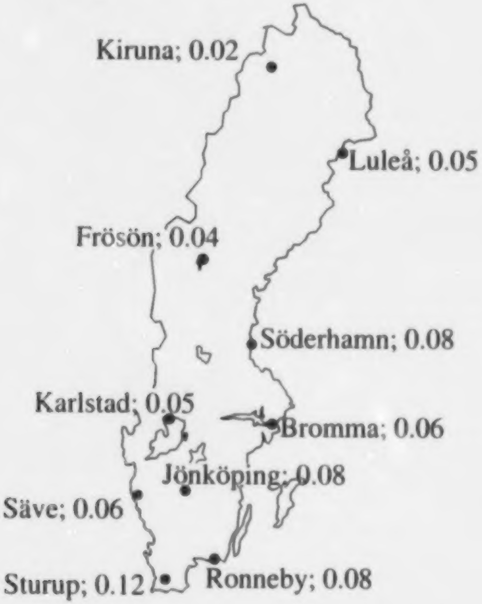

Fig. 6: Correction term for various places in Sweden. For practical purpose, we can use 0.10 in southern Sweden and on the coast, and 0.05 in the rest of the country

6 Conclusions

The examples show that the variations in climate from one year to another produce differences in the results of the calculations. When designing a structure, allowance can be made for the extreme years by taking a reference year and then applying an appropriate correction factor. Each type of moisture condition and design requires knowledge of the appropriate reference year and correction: in some cases, a correction term is more suitable, while in others it is a correction factor that is to be preferred. The difficulty to find common reference years and corrections for different moisture conditions and designs makes it sometimes easier to perform the calculations for a long (30 years) suite of real years and select the extreme values from these results.

7 References

Harderup, E. (1998) Metoder att välja korrektioner vid fuktberäkningar med variabelt utomhusklimat. (In Swedish.) [Methods of selecting corrections for moisture designs to deal with variable outdoor climate.] Report TVBH-1011, Lund University, Lund, Sweden.

Nevander, L.E. and Elmarsson, B. (1991) Fuktdimensionering av träkonstruktioner - Riskanalys. (In Swedish.) [Moisture protection of wooden structures - Risk analysis.] Report R38:1991, Swedish Building Research Council, Stockholm, Sweden.

87 COUPLED HEAT AND MASS TRANSFER IN BUILDING MATERIALS

A.N. Noumowe,[1] T. Ohkubo,[2] M. Makatayama,[2] and
K. Watanabe[2]

[1]Université de Cergy-Pontoise, France; [2] Production Engineering
Department, Building Research Institute, Tsukuba, Japan

Abstract

This paper presents a part of numerical and experimental studies carried out in BRI of Japan on multi-layered structures subjected to variable climate conditions The initially quasi-saturated specimens of different mortar with different exposure sides were dried under the variable climate conditions. The environmental temperature, relative humidity and gross weight of each specimen were measured continuously. The distribution of the temperature, relative humidity, moisture content distribution of each specimen was calculated using a two dimensional heat and moisture simultaneous model and the gross weight evolution calculated from the obtained moisture distribution was compared with the measured one for each specimen. The physical properties of each component material necessary for calculation were obtained through experiments executed in BRI, JTCCM (Japan Testing Centre for Construction Materials) and CSTB in France. The numerical model was developed in BRI for prediction of heat and moisture transfer in multi-layered building components at hygroscopic conditions. The good agreement between calculation and measurement shows the applicability of the model to the quasi-saturated conditions.

Keywords: Temperature, relative humidity, mortar, modelling, diffusivity

1 Introduction

The study of heat and mass transfers in building materials leads to a better understanding of constructions hygrothermal behaviour in service conditions. The knowledge of material properties is necessary to predict the behaviour of buildings linked with their durability. Structural elements can be subjected to variable room and external climate conditions that can cause damages. One of the main design aims is to prevent the materials deterioration.

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Building components like walls, in service conditions, are subjected to variable climate conditions which are different between their exposed sides. On the external face, the conditions are more or less severe due to temperature variation, solar radiation, wind, rain or other weathering conditions. On the internal face, the temperature is usually regulated for the inhabitants' comfort. Therefore, the building components are exposed to two different climates which can create a hygro-thermal and mechanical strains and stresses in the components. These strains and stresses can create damages that suffer the durability of the component. In multi-layered structures, there can be condensation that participate to deteriorate the components to a great extent.

Many building materials are porous and can keep moisture in the porous structure. In these materials, water can be in solid, liquid or gaseous phase. The phase changes and the migration of moisture are always linked to heat transfer. Coupling heat and humidity flux seems to be the best way to describe the building elements behaviour.

Numerous studies showed the difficulty of developing models and measuring model parameters. Among coupled heat and mass transfer models, Philip and De Vries model is the most commonly used one. It is a generalisation of the Darcy's law and is an extension of the Fick's law to porous media. D. QUENARD (1989) proposed a method to determine the transfer coefficients for the model from the micro-structure of the material.

Experimental and numerical studies were carried out in BRI (Japan) on simultaneous heat and mass transfers in building components. On the basic studies carried out by M. Matsumoto, K. Watanabe and Y. Sakamoto (1994) developed a computer program of two-dimensional heat and moisture transfer applicable to the multi-layered building components. An external wall can be composed of different materials like concrete, mortar, gypsum board, mineral wool, wood fibre board, air layer, vapour barrier, etc. This program permits the prediction of the distribution of temperature, humidity and moisture content in the components exposed to two different variable climate conditions.

This paper deals with one application of this computer program: the simulation of drying experiments performed in laboratory on initially saturated specimens. The study included two parts:

- Tests were carried out on mortar specimens in different drying conditions.
- Modelling of the hygric behaviour of the specimens was carried out. For modelling purposes the computer program HMTrans was used. HMTrans permitted to determine time-dependent distributions of temperature and moisture content in multi-layer structures during wetting and drying. To validate the computer program HMTrans, drying of mortar specimens was simulated. Numerical results were compared to experimental results.

2 Governing equations

The main process of the macroscopic modelling is presented. The modelling is based on heat and moisture balance in the process of moisture migration. The migration of moisture in a porous material is usually due to the gradients of moisture content and temperature. So the moisture existing in a certain cell migrates to the surrounding cells according to the difference of moisture content and temperature. In this simulation we suppose that there is no chemical reaction which occurs in this migration process and the local moisture content is determined by the sorption isotherm of the material as defined by equation (3). If the migration in at least one direction can be neglected, the moisture content change of the cell in a unit time is determined by the diffusion of moisture in two directions as shown in equation (1). The moisture migration is coupled with heat transfer and the temperature change of the cell in a unit time is determined by heat loss or gain due to heat conductance and potential heat in migrated water as shown in equation (2).

$$\rho_w \frac{\partial \phi}{\partial t} = \frac{\partial}{\partial x}\left[D_\phi \frac{\partial \phi}{\partial x} + D_T \frac{\partial T}{\partial x}\right] + \frac{\partial}{\partial y}\left[D_\phi \frac{\partial \phi}{\partial y} + D_T \frac{\partial T}{\partial y}\right] \tag{1}$$

$$\rho c \frac{\partial T}{\partial t} = \frac{\partial}{\partial x}\left[\left(\lambda + \gamma D_{TG}\right)\frac{\partial T}{\partial x} + \gamma D_{\phi G}\frac{\partial \phi}{\partial x}\right] + \frac{\partial}{\partial y}\left[\left(\lambda + \gamma D_{TG}\right)\frac{\partial T}{\partial y} + \gamma D_{\phi G}\frac{\partial \phi}{\partial y}\right] \tag{2}$$

$$\phi = F(h,t) \equiv F(h) \tag{3}$$

ϕ : volume water content,
ρ_w : liquid water density,
h : relative humidity,
γ : vapour/liquid change heat of water ,
D : water diffusivity,
ρ : material density,
λ : material thermal conductivity,
T : temperature,
c : material specific heat,
t : time,

subscripts
T : temperature gradient,
G : gas phase,
ϕ: moisture content gradient.

Equations (1) and (2) can not be applied to non-hygroscopic materials. For surfaces in contact with non-hygroscopic materials or air layers, the equations (1) and (2) should be replaced by (4) and (5). The boundary conditions are also replaced by $\phi = f(X, T)$.

$$-\left(D_\phi \frac{\partial \phi}{\partial x} + D_T \frac{\partial T}{\partial x}\right)s = \alpha'\left(X_N - X_S\right) \tag{4}$$

$$\left\{\left(\lambda + \gamma D_{Tg}\right)\frac{\partial T}{\partial x} + \gamma D_{\phi g}\frac{\partial \phi}{\partial x}\right\}s = \alpha\left(T_N - T_S\right) + \gamma \alpha'\left(X_N - X_S\right) \tag{5}$$

X : absolute humidity,
T : temperature,
α : thermal conductivity of the non-hygroscopic material,
α' : hygric conductivity of the non-hygroscopic material,
S : surface of the hygroscopic material,
N : surface of the non-hygroscopic material.

In the present model, the following assumptions were considered in the hygroscopic region.

$$D_\phi = D_{\phi g} = \lambda' p_{vs}\left(\frac{\partial h}{\partial \phi}\right)_T \tag{6}$$

$$D_T = D_{T_g} = \lambda' h \frac{dp_{vs}}{dT} \tag{7}$$

λ' : vapour permeability,
ϕ : water content in volume %,
T : absolute temperature.
p_{vs} : saturation vapour pressure,
h : relative humidity,

Such switching is quite inconvenient for making a practical model applicable to both hygroscopic and non-hygroscopic materials. In the present model, water chemical potential was considered as driving force for hygric transport in order to apply the diffusion equation to both liquid and vapour phases (water chemical potential is continuous even if there is a phase change without chemical reaction). By using the potential μ instead of the water content, equations (1) and (2) become :

$$E\frac{\partial \mu}{\partial t} = \frac{\partial}{\partial x}\left(AX\frac{\partial \mu}{\partial x} + BX\frac{\partial T}{\partial x}\right) + \frac{\partial}{\partial y}\left(AY\frac{\partial \mu}{\partial y} + BY\frac{\partial T}{\partial y}\right) \tag{8}$$

$$F\frac{\partial T}{\partial t} = \frac{\partial}{\partial x}\left(CX\frac{\partial T}{\partial x} + DX\frac{\partial \mu}{\partial x}\right) + \frac{\partial}{\partial y}\left(CY\frac{\partial T}{\partial y} + DY\frac{\partial \mu}{\partial y}\right) \tag{9}$$

μ = chemical potential
ϕ = equilibrium water content,
A = λ'_μ = moisture diffusivity due to chemical potential gradient,
B = λ'_T = moisture diffusivity due to temperature gradient,
$C = \lambda + \gamma\lambda'_{TG}$, $D = \gamma\lambda'_{\mu G}$ $E = \rho_w \dfrac{\partial \phi}{\partial \mu}$ $F = c\rho$

By introducing the chemical potential, the coupled heat and moisture transfer is thus simplified into two partial differential equations where temperature and water chemical potential are used as driving potentials for both heat and moisture transfer. Necessary material parameters are moisture diffusivity and thermal conductance together with coefficients that determine the capacity of moisture and heat. The materials thermal conductivities based on chemical potential can be obtained as follows :

$$\lambda'_\mu = \lambda'_{\mu G} + \frac{h}{R_v T}\frac{\partial \phi}{\partial h} D_{\mu L} \qquad \lambda'_T = \lambda'_{TG} + D_{TL}$$

$$\lambda'_{\mu G} = \frac{P_{vs} h}{R_v T}\lambda' \qquad \lambda'_{TG} = \frac{dP_{vs}}{dT} h \lambda'$$

Materials hygro-thermal properties are recorded in a data file. Equations (8) and (9) determine temperature and potential distribution in the related building components. Then,

- relative humidity is converted inversely as RH = exp (μ / R_v T),
- moisture content is obtained through sorption isotherms (equations of state) as a function of RH,
- vapour pressure is obtained through saturation vapour pressure curves and relative humidity as $p_v = RH.p_{vsat}$

For each material, the following properties are recorded in the data file (density, specific heat, thermal conductivity, vapour permeability and sorption isotherms). There are not enough data on the physical properties of the building materials. For our calculations, material properties are determined experimentally either in BRI Production Engineering Department or in CSTB-France by D. QUENARD.

Moisture content in the specimens during drying was measured and compared to numerical results obtained with HMTrans. For 3 drying conditions, temperature and moisture distributions as functions of time in mortar specimens under unstable climate condition were analysed.

3 Experimental procedure

Ordinary mortar specimens were dried in laboratory conditions. Room temperature and relative humidity were recorded during the whole drying process. The temperature oscillated between 18 and 24 °C while the relative humidity oscillated between 27 and 49 %. These data were used in modelling as input. 40x40x160 mm specimens were sealed on 2 or 4 faces in order to permit drying in 1 or 2 directions. The specimens were of type A, B and C (figure 1). At the beginning of the drying, the specimens were water saturated.

A-prismatic specimens were sealed on the two small faces. Drying took place on the four large faces. Bidimensional drying permitted to test the 2D program. B-prismatic specimens were sealed on 2 large faces and on 2 small faces. Drying took place on two opposite large faces and was one dimensional. C-prismatic specimens were sealed on the four large faces. Drying took place on the two opposite small faces and was one-dimensional. C specimens dried very slowly. B and C specimens permitted to test the computer program for different drying kinetics.

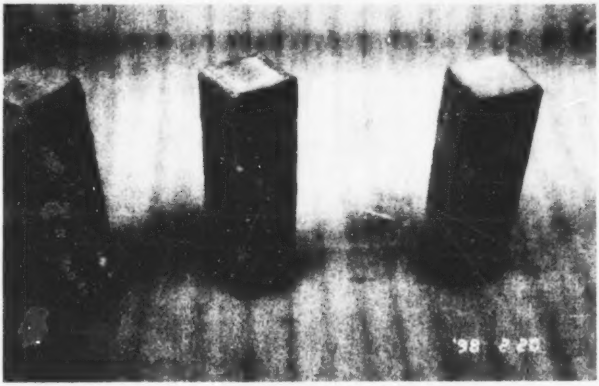

Fig. 1: Tested specimens (A, B and C types)

4 Numerical procedure

Simulation of the coupled heat and mass transfer in building components requires detailed knowledge of the corresponding thermal and hygric transfer parameters. Tests were performed in order to determine the properties of the mortars used. HMTrans provided default values for thermal convection between materials and air (K. Watanabe, 1994). The modelling included drying in one or two dimensions.

Mortars specimens were modelled as multi-layer structures. For example, C specimens were modelled as 16 layers of materials. The dimensions of each layer were 40x40x10 mm. In this example, the material (ordinary mortar) is the same in all layers.

The main features of the HMTrans program were as follows:

- A two-dimensional finite difference method was used,
- Moisture as well as temperature distributions were calculated in a transient way,
- Convection transfer of heat and moisture was not considered except at the boundaries,
- New materials properties may be added to a materials data file,
- A preprocessor eased the generation of input (environmental climate, structure geometry,...)
- The program was written in Fortran and run on a compatible PC,
- The program permits the analysis multi-layered building components.

5 Experimental and numerical results

The changes in weight of A1, A2, A3 and A4 specimens are plotted on figure 2. The weight is plotted in reference to each specimen initial weight. The model result is shown as well. The four curves are very similar. Hygric equilibrium was achieved in A specimens after 30 days. The numerical curve is very close to experimental curves. Drying was completed after about 30 days.

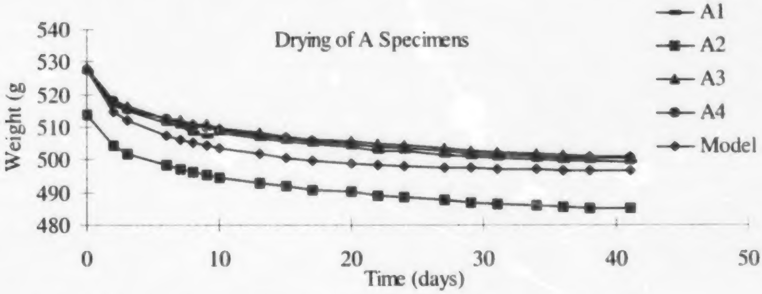

Fig. 2: Weight variations of A1, A2, A3 and A4 specimens compared to modelling result

Figure 3 shows the weight variations of B specimens compared to the model result. Drying was completed after more than 40 days. Figure 4 shows the weight variations of C specimens. Experimental results C1, C2 and C3 are compared to model result. Figures 5 to 8 show numerical moisture field in C specimens during drying. Due to the symmetry, the relative humidity field is presented only for the specimen upper half. During the first two days, the specimens were predried. Figure 5 shows the initial moisture content at the beginning of the drying process. The surface relative humidity was 40 %. At the centre of the specimen, the relative humidity was 98 %. During the first 10 days of the drying process, the relative humidity at the centre of the specimen decreased from 98 to 88 %. Tests were carried out during 40 days. The modelling was carried out for the first 40 days of drying.

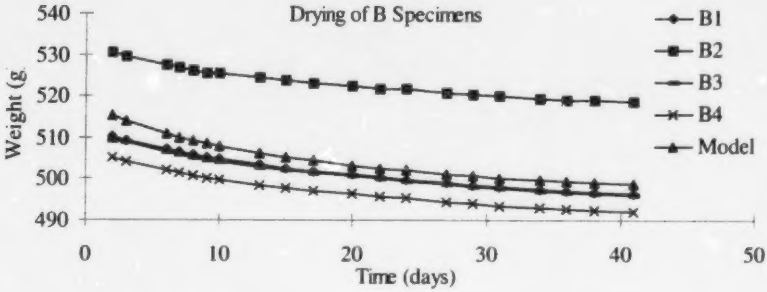

Fig. 3: Weight variations of B1, B2, B3 and B4 specimens compared to modelling result.

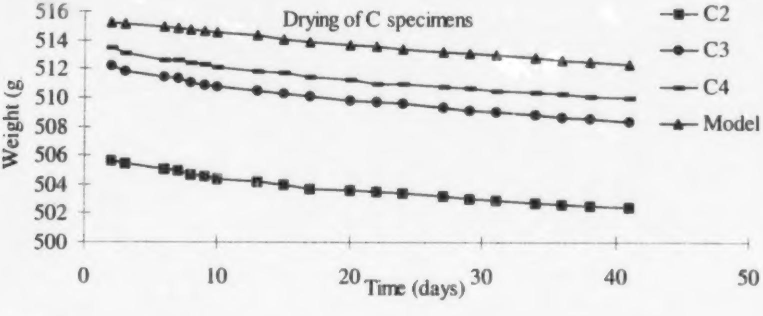

Fig. 4: Weight variations of C2, C3 and C4 specimens compared to modelling result

Forty days after the beginning of the drying process, the relative humidity at the centre of the C specimens was 83 %. C specimens dried very slowly because the drying surface was very small. For experimental and numerical weight curves, the drying rate was very low. The results simulated for moisture losses and drying rates came fairly close to those found in laboratory measurements. These results permitted to validate the modelling.

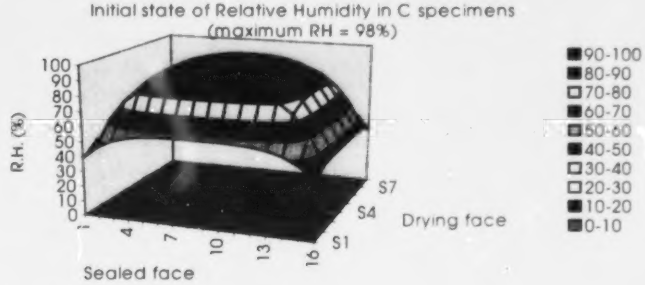

Fig. 5: Initial relative humidity of C specimens

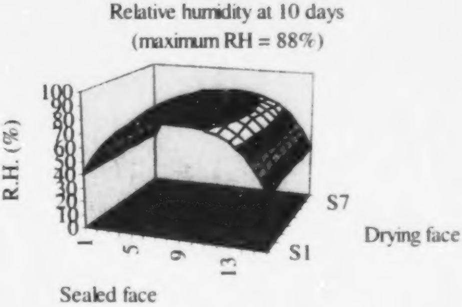

Fig. 6: Relative humidity 10 days after the beginning of drying process

HMTrans computer program permitted the prediction of moisture field during drying process. The calculated water content decreased at the same rate as the experimental water content. Modelling results were in good agreement with experimental results. The comparison permitted to validate not only the governing equations and parameters but also the measured hygrothermal properties of the materials used.

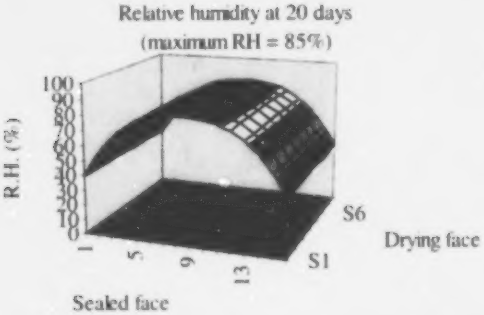

Fig. 7: Relative humidity 20 days after the beginning of drying process

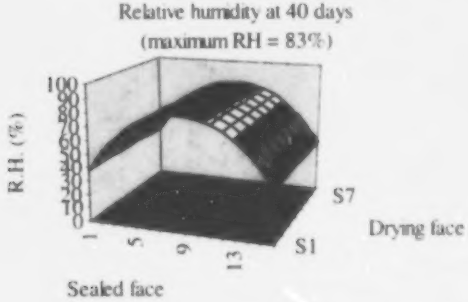

Fig. 8: Relative humidity 40 days after the beginning of drying process

6 Discussion

The way the computer program was built made it easy to include, in the future, the effects of hysteresis in the sorption curves. Pedersen (1990) showed that the hysteresis was not so important in the analysis of "real" construction subjected to a naturally varying climate. This author showed that the use of a middle curve, between the drying and wetting curves in the diagrams for moisture retention, gave practically the same results as for using the hysteresis.

K. Watanabe et al (1995) developed an innovative measuring procedure in order to enable fast determination of moisture transmission coefficient in building materials without any special equipment. An approximating equation gave the actual water content w in function of the both relative humidity and the saturation water content $w = w_s \, RH^a \exp[b(1-RH^c)]$. For building materials, one curve was fitted for adsorption and another one for desorption. Though, the hysteresis can be accounted for. This method was applied successfully by Watanabe and Tada (1997). The approximating equation can be added to the computer program.

7 Conclusion

Time-dependent distributions of temperature and moisture content in mortar specimens during drying were determined. The application of the model on mortar specimens showed its capability.

Initially quasi-saturated mortar specimens were dried under the variable climate conditions. Prismatic specimens were sealed on 2 or 4 faces in order to permit one-dimensional and bidimensional drying. The environmental temperature, relative humidity and gross weight of each specimen were measured continuously. The distribution of the temperature, relative humidity, moisture content distribution of each specimen was calculated using a two dimensional heat and moisture simultaneous model. The good agreement between calculation and measurement shows the applicability of the model to the quasi-saturated conditions.

The results of the proposed numerical model, which was based on a generalised formulation combining heat and mass transfer trough porous media indicated that the mutual dependence between the transport of heat and moisture was taken into account. The ability of the computer program to predict moisture content and moisture distribution accurately was verified. This has proven to be a computer program that gives sufficient accuracy considering the uncertainty in material parameters. Of course, the accuracy of computer programs such as the one developed depends strongly on the materials properties fed into them.

8 Acknowledgement

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9 References

Mihashi H., Numao T. (1995) Influence of Material Structure and Temperature on Moisture Migration and Shrinkage in Hardened Cement Paste and Mortar. International Workshop on Mass-Energy and Deterioration of Building Components. Paris.

Ohkubo T., Makatayama M., Nemoto K. (1998) Flexural Deformation of Concrete Slabs Utilising Casting Forms Under a Continual bending Load. Workshop on Modelling of Deterioration in Composite Building Components due to Heat and Mass Transfer. BRI.

Pedersen C.R. (1990) Combined Heat and Moisture Transfer in Building Constructions. Thermal Insulation Laboratory. Technical University of Denmark. Report N 214.

Quenard D. (1989) Adsorption et Transfert d'Humidité dans les Matériaux Hygroscopiques. Thèse de Doctorat de l'Institut National Polytechnique de Toulouse.

Watanabe K. (1994) Hygrothermal Condition in Building Components Simulated with Heat and Moisture Simultaneous Transfer Model. Building Research Institute, Japan.

Watanabe K., Ashie Y., Shiligersky I. (1995) Towards Practical Simulation of Hygrothermal Behaviour for Building Components. Part 2 Measuring Moisture diffusivity in Porous Building Materials for Heat and moisture Calculations. BRI.

Watanabe K., Tada S. (1997) Rapid Measurement Methods for Moisture Transfer Properties. International Conference of COST Action E8 Wood Mechanics. Copenhagen, Denmark.

88 MAPPING RAINFALL DISTRIBUTION CHARACTERISTICS ON FACADES USING SURFACE DEPOSIT GEOMETRY

P. Snape and B. Atkinson

UNITEC Institute of Technology, Auckland, New Zealand

Abstract

The three most relevant kinds of precipitation that effect the performance of a building facade are described as 'direct', 'splashback', and 'runoff'. The determination of the intensity of direct rainfall on a facade is seen as the first step in understanding the characteristics of durability and performance of an external envelope.

Away from the facade other indicators have been devised such as the 'driving rain indices' by the NBRS, BRE and others. These can be used as a measure of severity at regional or local scales. In the field, and at the surface, vertical plate gauges can be placed at strategic locations and long term measurements estimated. There is generally a good correlation between the surface distribution of direct rain on a facade and the extent of maintenance problems, including material degradation, failure and appearance, as well as internal damage by water penetration. The long term nature of vertical plate gauge measurements detracts from its usefulness.

This paper reviews methods of measurement of direct rainfall distribution at various scales. It introduces a method based on the geometry of surface deposits which allows a rapid comparison of facade performance, and indicates likely trouble spots.

Keywords: facade performance, rainfall distribution, surface deposit mapping.

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1 Introduction

Studies of rainfall distribution on building facades forms part of the general research work related to the 'performance equation' that is carried out at UNITEC. (Atkinson 1997, 1998) has outlined the philosophy that is followed, leading up to this and other studies of material performance. In this paper one facet of the causal factor 'environment', that of direct rainfall, is of special significance.

For most external materials the rainfall absorption curves follow an inverse exponential function with respect to time. If the intensity of rainfall exceeds the gradient of this curve a rainfall 'run-off' will occur. For a given constant intensity (mm/hr.) it is only a matter of time before a run-off occurs at the surface. The period which elapses between the rain first reaching a surface, and the occurrence of a run-off, we refer to as the 'interception period'. The interception period is important because of its influence on the probability of frost or moisture attack, and the incidence of a run-off, which apart from cleaning a surface, results in local concentrations of water at joints and junctions. Investigations into these effects are legend. (Parker 1966) gave typical mass saturation curves for rendered and unrendered brickwork. These follow a recognisable saw-tooth pattern; the peaks representing the intervals of time between maximum saturation. The main difference is that the input is at indeterminate intervals rather than at regular intervals, and a slow build-up of moisture within the wall, or a slow evaporation can occur responding to the statistical chatter of the weather and other causal factors.

In connection with the wetting of external surfaces there is a close relationship between rainfall and wind independent of their separate effects. Except for a splashback at ground level, and on the surface of projections, no rain reaches a vertical surface in still air. On an open site for a given gradient height Hellman discovered the variation in wind speed with height according to a power law. The preceding example, for instance, presupposes the existence of a wind; and whilst this is generally true for most of the time, the directional frequency of wind at times of rain may not be the same as the directional frequency of wind at all times. Mean wind speeds during rain are greater than mean wind speeds taken over all times. (Prior 1985) has verified earlier workers' results which give UK factors varying from 0.9 to 1.5.

More important therefore than either wind or rain is the combination of both which is generally known as driving-rain.

2 Driving-rain

History has shown that the results of general field surveys, or specific surveys such as those of paint failures on timber substrates, emphasize the importance of moisture in the deterioration of external materials. The amount of driving-rain reaching a surface is nearly always the main criterion governing the rate of deterioration of materials and design elements. It also governs the amount and distribution of foreign particles, such as street dust, pollutants, and organic matter that adhere and accumulate on external surfaces; and thereby to a large extent is an important factor effecting external changes in appearance as well as composition.

(Hoppestad 1955) originally defined an index of driving-rain as the product of the mean wind speed (m/sec) and rainfall (mm). He drew a contour map for Norway which showed the relative severity of driving-rain in different parts of the country. Several maps have been devised for Denmark, Canada, and the UK (BRE and Met. Office). In calculating the driving-rain in this way an assumption is made that the quantities of rain reaching a surface are proportional to the index value. The index, currently modified in various compensatory ways to take into account terrain, building shape, and aspect, is essentially a relative geographical measure of severity rather than an exact measure. It can therefore be omni-directional or directional, regional or microclimatic.

2.1 Vertical gauges

More exact measures of driving rain using vertical raingauges (R_v mm), have been made by various researchers in the past working in a number of fields. (Pers 1932) designed a four-way 'vectopluviometer', which is essentially the same type of gauge used later in Norway, known as the NBRS gauge. (Fourcade 1942) used an elaborate four-way vertical gauge for estimating driving-rain and condensates at ground level. The gauge was designed for agricultural and forestry use. An octagonal or eight-way vertical gauge is favoured in the UK. The first of these was built in 1936 at the Building Research Station by Beckett (Type DR1), and has been used by (Lacy 1951,1964). (Hamilton 1954), in Candada, used a variety of tilting and directional vertical gauges in an attempt to simplify catchment measurements in forest areas. In all these cases the gauges, set with the vertical apertures facing the cardinal points, are placed away from buildings on open ground. In 1959 the first measurements of driving-rain were made in Denmark using an omni-directional 'dish' gauge. (Korsgaard and Madsen 1962) have described this gauge, which does not measure the quantities of rain falling on the external walls, but allows closer approximations to be made. Later computations by (Prior and Caton 1985) concluded that the earlier DRI's were marginally too low. The correlation between computed driving-rain indices and the vertical catches of most of these gauges is fairly good on open sites, and has been found to be linear. Being a product of variables, DRI's unfortunately are not additive. The sum of the directional DRI rose indices does not equal the omnidirectional index. Nor does the highest DRI relate to the prevailing direction of the wind-with-rain. Most gauges include a horizontal aperture (R_H mm) which allows estimates of the rainfall angle of incidence ($\arctan(R_v / R_H)$) to be calculated near ground level. A map showing these roses was incorporated into B.R.S. Digest 127 (1971). British Standard 8104 : (1992) builds on much of the above early pioneering work.

It is clear that a directional driving-rain rose is necessary at the correct scale and microclimatic location, if a reasonable correlation with general field survey results of external deterioration is to be made. Unfortunately these are not available at microclimatic levels, but indicators can be deduced from urban area DRI values, where meteorological measurements quite often exist outside Prior's Master Met. stations. If these include rainfall intensity measurements then computer processing can yield satisfactory DRI working tools. A 'walkabout' around a locality and site often reveals as much, if not more microclimate information.

The total or omni-directional driving-rain index (DRI) for the Manchester area UK, where most of the research work for this paper was originally carried out, is based on a mean annual rainfall of 805 mm, and a mean wind speed of 5.20 m/sec, giving a DRI of 4.18 m^2/sec. yr. This value, with a suitable K value (Prior 1985), corresponds to the value given on the meteorological office DRI contour map.

The driving-rain directional roses shown in Digest 127 (BRE 1971) were based on three categories of precipitation and Beaufort wind speeds: for this reason weighting factors had to be used which were roughly proportional to the intensity of rainfall and mean wind speed. Thus estimates of actual quantities of rain reaching a vertical surface were not made. An alternative method was adopted for the Manchester area surveys based on a Monte Carlo sampling of wind-with-rain data.

2.2 Rainfall intensity, droplet size spectrum, and directions

If the mean wind speed for any direction was constant; and providing rain fell for the same proportion of time from each direction; and with equal intensity; then the driving-rain rose could be represented simply by the normal directional frequency wind rose.

The quantities (R_s) and mean incident angles of rain (i) can be computed by using a 50%tile volume median raindrop terminal velocity (v_t) based on the mean intensity of rainfall (R_h), and using equations given by many authors dating back to (Laws and Parson 1943), (Gunn and Kinzer 1949), (Best 1950), and (Berry and Pranger 1974).

It is important to note that the estimates of driving-rain derived from regional data considerably over-estimate quantities actually falling on a wall because of momentum and aerodynamic flows. (Lacy 1964) found, for example, that only 10%-25% of computed driving-rain reached a test wall because of the effect the wall had on wind flow, and because of the sheltered site. The computed figures based on synoptic station data refer to wind speeds 10 m. above ground level on an open site, and to rainfall catches on the unobstructed ground. The estimated amounts using this approach represent the amounts that would pass through an imaginary mesh surface 10 m. above ground level. For Garston, Lacy found that the most exposed walls of a two-storey house received about 25% of the computed driving-rain, and the most sheltered walls about 6%. The quantities of rain falling on a flat roof are about the same as on the ground nearby. The windward slope of a pitched roof receives proportionally more rain per m^2 than the lee slope, with some adjustment for wind flow effects. The total quantity of driving rain that falls on a building is equal to what we call the 'rainfall sink' about a building shown as isohyet contours (Fig. 1).

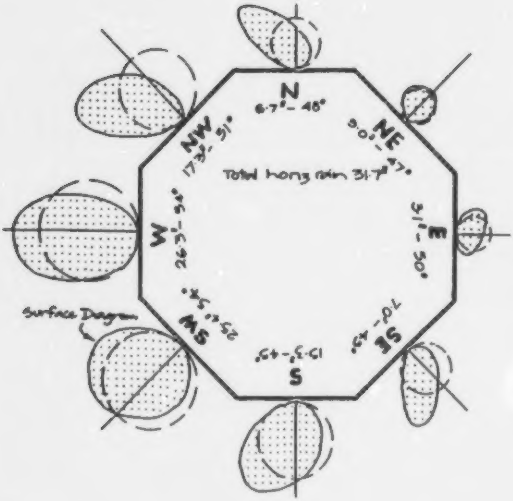
N
6·7"– 48°
NE
9·0"– 47°
Total horiz rain 31·7"
E
3·1"– 50°
SE
7·0"– 49°
S
13·3"– 49°
SW
23·4"– 54°
W
26·3"– 54°
NW
17·3"– 51°
Surface Diagram

Fig. 1: The rainfall sink

2.3 Rainfall source and vector diagrams

A useful tool was invented to assist in our research. As an additional aid to site investigations of material deterioration, a rainfall 'vector ' diagram was devised which indicates the proportion of rain falling on a given wall between two specified directions or poles. The directional diagram is illustrated in (Fig.2).

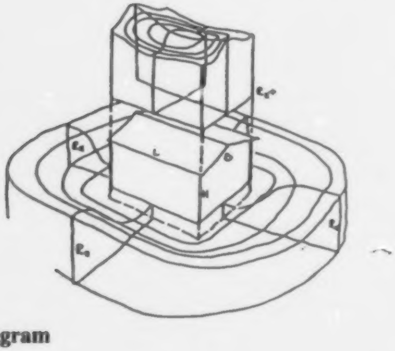

Fig. 2: DRI Vector Diagram

The normal component of driving rain falling on a surface is equal to R_H tan i sin ø, where R_H is the normal horizontal catch, i is the incident angle of the rain to the vertical and ø is the horizontal angle of the rain to the wall. By summing the normal components of the driving rain over 180° the total D.R.I. for any wall facing a given compass point can be estimated. For the vector diagram, five percentage vector values for each of the eight compass directions, averaged over equal segments of 180° exposure, were plotted from an arbitrary origin; and an approximate curve was sketched in. The resulting closed area bounded by two vectors was called a rainfall vector diagram. The proportion of the total possible rain falling on a surface between two specified directions is thus equal to the area of the vector diagram enclosed by the two poles. Each diagram, for each direction, has an area proportional to the total vertical direct rain driving onto its surface.

The diagram may be used to estimate the effects of curtailment or obstructions on the quantities of rain reaching a given point on a surface. For example, from any given point in a reveal, two curtailment lines can usually be drawn on plan from the point itself to the external corners of the reveal. By placing the origin of the appropriate surface diagram on the point, and correctly aligning the base line of the diagram with the wall, the area enclosed by the two lines can be estimated as a ratio of the total area of the diagram. The product of this ratio and the total estimated driving-rain for that orientation, yields the total amount of rain reaching the chosen part of the wall. The method is illustrated in (Fig. 3).

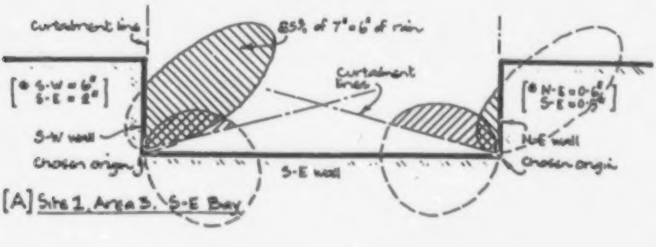

Fig. 3: Curtailment lines and vector surface diagrams

The vector diagram is most advantageously used with each directional diagram drawn separately on squared transparent plastic. In this way two diagrams can be overlaid, and placed say in the corner of a reveal: the differences which occur in the quantities of driving-rain reaching either bounding surface can be estimated and compared with site observations. The southern reveal of a window which faces south-east, for example, may receive almost negligible amounts of rain compared with the opposite reveal. The diagrams shown in this work are based on estimates of relative quantities of driving-rain only, as an aid to assessing external performance. Current work has been directed towards more accurate computer estimates.

3 Rainfall distribution on buildings

Perhaps the most unusual aspect of building design is that there is a dearth of information on the distribution of rain over or around a building (Fig. 1). External design detailing is not usually based on fundamental knowledge of surface water flow, because such a body of knowledge is hard to find. This is probably because the principles of design detailing are basically traditional; represented by a few simple economic design rules handed down to successive generations. Unfortunately whilst these details may protect the interior from moderate extremes of suburban climate, trouble arises when they are used indiscriminately above two-storey level, or in places of severe exposure at lower levels.

There is also a dearth of measurements of driving-rain taken on actual buildings, but from as early as 1937, various attempts have been made to devise suitable wall gauges in order to judge distributions. (Sandberg 1968) estimated building centreline wall distributions based on wind vectors derived from a wind tunnel, and computer calculations of droplet flight paths. (Rogers 1974) repeated the exercise, but in both methods the results do not fit our field observations of surface deposits - the problem probably deriving from the adopted raindrop spectrum and the wind vector fields.

Flat rectangular plate gauges with a raised rim and collecting channel have been used by the Building Research Station for some years. This type, originally designed by (Holmgren 1937) (Trondheim) has been used by the Building Research Station on buildings at Garston, Thorntonhall and Glasgow. Other types of gauge have been designed, such as the circular pot type due to (Croiset 1959), and various recessed gauges. BRE have undertaken work with flat copper plate gauges with raised rims and collecting bottles.

If sufficient evidence were available the results of measurements made with wall gauges would show means and extremes of driving-rain; the distribution of rain over various elevations; and allow correlations to be made with computed driving-rain, and with, for example, free standing octagonal gauge measurements. Lacy correlated the results of all three methods of estimating driving-rain using gauges, fixed to a 2.8 m high testing wall on the B.R.S. exposure site. The results of five years exposure indicated that the total driving-rain on the wall varied between 10% and 25% of that derived from nearby standard meteorological measurements. Theoretically the greatest quantities of rain should fall on the corners and edges of buildings where the streamlines are known to close, and larger raindrops (spectrum domain approx. $0.1>d_r>6.0$) probably shatter before impact because of inertia and centrifugal forces.

It is evident that measurements made so far using wall gauges are of limited use to the designer in respect of a particular building because of the long term nature of the measurements; but the general principles of driving rain have been established. Minimum exposure times of several years are generally considered necessary in meteorological work of this kind if reliable averages are to be obtained, and low probability events recorded.

3.1 Surface deposits and their geometry

General observation of many buildings will reveal striking differences in external appearance due to pollution deposits. It was noted that the staining and suface deposit 'fringes' (unwashed areas) below projections varied in depth from place-to-place on the same elevations, as well as on different elevations. It was conjectured that a useful relative measure of averaged rainfall distribution over such a surface might be found by comparing the depth or width of deposit fringes on different parts of an elevation providing estimates of ø (angle of wind to the wall) could be made. It was also necessary to examine the relationship between the incident angle i of the average median diameter droplet and the depth of the cut-off line below projections.

The cut-off line, or bottom edge of most deposit fringes is not a definite line. There is usually a gradual change from black or brown to the self colour of the material. Below the eaves of several survey buildings on west elevations in our survey area, and on low absorptivity surfaces, the gradation occurred over 10 to 20 mms of wall surface. On the same type of surface, facing south and east an increase in the width of the cut-off band was observed, which indicated a decrease in the mean incident angle. The cut-off band width will vary according to the solubility of deposits; absorptivity and texture of the material; the normal incident angle i of the median droplet; and the frequency of rain from different directions (variations in ø). If an ogive rainfall distribution is assumed over the band width, the incident angle of the median diam. droplet can be assumed to lie close to the bottom of the band limits. The amount of rain falling on a wall for low values of ø (< 30°) *is* also small compared with the total amounts of driving-rain which strike at greater angles. This is particularly true for west elevations in the Manchester survey area, but probably not for north-west or south-east elevations (Fig. 2).

The amount of rain fàlling on a wall varies according to droplet interaction with wind vectors upstream, and to streamline forcing flows leading the droplet either around or against the surface. An easily recognised overall pattern of surface deposits can be found on flat elevations which have regular relief. Even so, microclimatic variations in driving-rain effect these patterns close to mullions, projecting structural members, canopies, and around the exposed edges of an elevation. It was concluded therefore, that the most suitable building for initial examination would be one having a fairly flat major elevation facing north-east or east. About equal amounts of rain fall from all directions for these orientations. Ideally, the deposit fringes should be measured away from nearby surface geometry projections, preferably on grid points which correspond with perhaps a repeated infill bay or window sill. The wall surfaces should be slightly absorbent.

. A building was found in Stockport, a little to the south of the survey area, which was suitable for study. It was about four years old at the time of inspection. It had an exposed, fairly flat, east north east elevation, and was clad in precast concrete units below sill level as shown in (Fig. 4). Measurements of the depth of the deposit fringes below each sill were made at a point to the left of the centre of each bay of the elevation. The nearest building, situated adjacent and to the north, was about 11m. away. It was about half the height of the one chosen for study, and probably affected the air flow around it.

A photographic technique was used to measure the depth of the sill fringes, using 35 mm projector slides. Both coloured and black and white slides were used.. Deposit fringe depths were measured on the screen image and adjusted for photo-perspective distortion. The results were entered in a matrix, each element of which corresponded to a measuring point. Empty elements were left around the edges, and were later filled with extrapolated values after the matrix was adjusted for errors along rows and columns. An assumption was made during iterative smoothing that the fringe depths formed a continuous three dimensional plane. In other words, the variations in rainfall distribution were smooth and continuous across the elevation. This is probably true on average, for a parallel plane 500mms or more off the elevation, but streamline distortion occurs around some vertical projections and other obstructions where these are large enough to cause surface turbulence or curtailment.

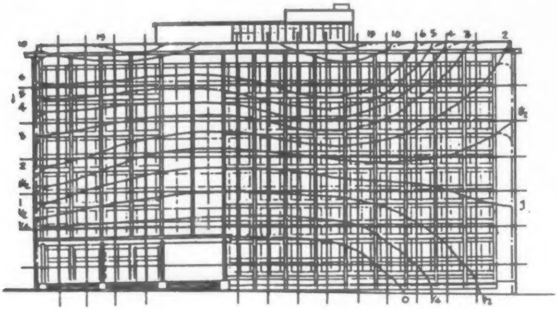

Fig. 4: The typical ground level precast concrete panel

At this stage an interesting discovery was made. It was found that the depth of the deposit fringes (d) varied logarithmically with the height (h) above ground level, and differently along each grid line, according to a law of the form :

$$h_0 - h = a.\log_e (w/d) - C \qquad \textbf{(1)}$$

where h_0 = the roofline height (m), w = sill projection (tan i = w/d) (m), C = effective height (m) for d = 0 , and a = surface location variable (m). This remarkably consistent effect may be related to Prandtl's logarithmic law for neutral or adiabatic equilibrium in the boundary layer wind flow. It shows the importance of flow in the far field as well as the near or surface flows.

It was also found that the incident angle of the driving rain corresponded to the regionally computed angle (mesh surface) shown on the vector diagram at a point in the centre of the maximum windward pressure zone, about 0.6 of the h_o for this building. Estimates of the distribution of the driving-rain (R_v) are mapped in contour form in (Fig. 4). The isohyets are contoured by using the relationship $R_v = R_H$ tan i where R_H is taken as 81 mm and tan i is equal to the ratio of sill projection w to fringe depth d .

The contours (Imp.) show an increase in driving-rain towards the roof line with maximum values towards the centre of the block. The influence of the penthouse flat can also be seen. There is some doubt about the validity of the contour values close to the edge (Fig.4), but the general configuration appears satisfactory. A large lower central area receives virtually no rainfall. In this zone the rain is probably carried away from the building; an effect suggested by examination of the streamlines. Lacy suggested that walls facing north in the London area, an area having the lowest DRI in the UK, received about 1/16th of the normal rainfall. For the chosen building a computational check on the mean value of the rainfall surface contour height showed that the elevation received 29 mms on average, about 1/28th of the normal rainfall or 38% of the computed value using meteorological data. On the chosen elevation most of the rainfall run-off is cast away from the wall by the sills and sloping slabs; the remaining areas receive rain according to the above estimates.

For many buildings the rainfall run-off characteristics are equally important as those of driving-rain; for it is by this next process that driving-rain is redistributed over an entire facade as it flows (eventually) downwards creating splashback and super-drops. (Ishikawa 1949) adopted some aspects of the method described above, using Tokyo medium rise buildings as case studies. Some aspects of his findings confirm those found by us. He was at some disadvantage because of lack of computing support.

In 1985 our method was repeated on two CBD buildings in the city of Adelaide, South Australia, and yielded similar results. These correlated well with the observed material degradation, appearance and reports of rainfall leaks in the facades.

4 Conclusions

By measuring the staining patterns on external facades some indication of long term rainfall distribution has been obtained in a fairly simple practical manner. It utilises the visible effects produced by historical rainfall. The characteristics of failure patterns can be detected in this way, as well as changes in appearance.

It is important to note that the method is indirect because it does not use actual surface measurements of vertical rainfall. These generally take too long to gather

Some of the background research to this work over the last three decades has been outlined. A tool described as the rainfall vector diagram has been introduced.

5 References

Atkinson, B. (1998) Weathering and the performance equation. *Proc. International ASTM Symposium on the Use of and Need for Performance Standards in Architectural Conservation.* April 18 - 19, 1998 Atlanta , Georgia, USA.

Atkinson, B. (1997) The performance of building elements and materials: a framework for research and teaching. *Proc. AUBEA Annual Conf. Auckland, New Zealand* Vol. 1. pp. 45-69.

Berry, E. X. and Pranger, M. R. (1974) Equations for calculating the terminal velocities of water drops. *J. Appl. Meteor.* vol. 13, pp. 108-113

Best, A. C. (1950) The size distribution of raindrops. *Qu. J. Roy. Met. Soc.* 76, p. 16

BRE (1971) *An index of exposure to driving rain* Digest 127, Series 2.

British Standards Institute (1992).) *Code of practice for assessing exposure of walls to driving-rain.* BSI, London. BS 8104

Croiset, M. (1959) L'humidite des murs d'habitation. Etude des facteurs reels determinant cette humidite et recherche de methodes dessais en oeuvre. *Centre Sc. and Blg. Tech.* Cahier 237, pp. 27-54, Paris .(CIB gauge)

Fourcade, H. G. (1942) Some notes on the effects of the incidence of rainfall over the surface of uneven ground. *Trans. R. S. S. Africa*, 29(1942), Pt. 3, pp.235-254.

Gunn, R. and Kinzer, G. D. (1949) The terminal velocity of fall for water drops in stagnant air. *J. Meteor.*, 6, pp. 243-248

Hamilton, E. L. (1954) Rainfall sampling on rugged terrain. *Tech. Bull. 1096. US Dept. of Agric.*, Washington, December 1954.

Hellman, O.U. (1915) in Geiger, R. *The climate near the ground.* Harv. Press 1965.

Hoppestad, S. (1955) Slagregn I Norge. *Norges Byggforskningsinstitutt*, Rapport Nr. 13, Oslo.

Ishikawa, H. (1949) A fundamental study for the evaluation of conditions for the wetting of wall surfaces by rainwater. *Procs. Arch. Inst. of Japan*, Oct.

Korsgaard, V. and Madsen, T. L. (1962) A note on driving rain measurements. *Heat Insul. Lab. Tech. Univ.* Denmark. (unpublished).

--- , (1962) Apparatus for measuring and registering of driving rain . *Heat Insul. Lab. Tech.* Univ. Denmark. (unpublished).

---- , (1962) Correlation between measured driving-rain and computed driving rain *Heat Insul. Lab. Tech. Univ.* Denmark. (unpublished).

Lacy, R. E. (1964) *Driving-rain at Garston,U.K.* Res.Series, No. 50 Min.Tech., Lon.

Lacy, R. E. (1951) Observations with a directional raingauge. *Qu. Journ. Met. Soc.*, 77(332), pp. 283-292, April, 1951

Laws, J. O. and Parsons, D. A. (1943) The relationship of raindrop size to intensity. *Trans. Amer. Geophys. Union*, 24 , pp. 453-460.

Parker, T. W. (1966) The technology of maintenance. *Procs. Blg. Maintenance Conf. Edin.* paper 2. R. and D., MPBW, London.

Pers, M. R. (1932) Relations entre les donnes pluviometriques et les precipitations totales recuellis per un basin. Intro. a l'etude de la stereo-pluviometrie. *La Meteor* 8(1932).

Prior, M. J. (1985) *Directional driving rain indices for the UK - computation and mapping*. BRE Report, Dept. Envir. London.

Rogers, G. G. et al (1974) Theoretical predictions of raindrop impaction on a slab type building. *Build. Sci.* Vol. 9, pp. 181-190. London.

Sandberg, I. (1968) Driving rain distribution over an infinitely long, high building: computerised calculations. *Div. Blg. Tech. Lund Inst. of Tech.*, Sweden. (Trans. 2054. 1975. BRE) pp. 1-4.

Appendix A

The North-West main frontage

The North corner

The West corner

Typical ground level precast concrete panel

PART SIX

BUILDING FACADES AND COMPONENTS

89 DURABILITY OF WALL SYSTEMS CONTAINING HOUSEWRAP

E.J. Bosack and E.F. Burnett

Pennsylvania State University, University Park, PA, USA

Abstract

Since its introduction, some twenty years ago, the use of housewrap in residential and commercial applications has increased significantly. Most of the research into the use and performance of the various housewrap materials appears to have been of a proprietary nature as little information is available in the public domain. This paper addresses some durability issues regarding the use of housewrap within the building enclosure system. A survey of in-place conditions and installation practices in Pennsylvania was carried out and some of these results are reported. In addition, an experimental investigation was conducted to demonstrate the contribution of differential wind pressure to not only the housewrap but also the other elements in a typical residential wall system. The effect of wind, especially in service, is discussed. Some reservations are also expressed about the behaviour and durability of housewraps in general.

Keywords: Housewrap, Air Barrier, Weather Barrier, Durability, Installation, Wind

1 Introduction

Builders, especially house builders, face an increasingly complex business environment; one that involves more stringent building codes and more regulation, aggressive litigation and many new and, sometimes, innovative products. Innovation always has an impact—economically, practically and behaviorally. In some instances, the effects of using new products on performance are not completely understood, nor are the longer-term implications known. Consider, for instance, the material generically referred to as housewrap. Since its introduction, nearly 20 years ago, this

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product has undergone many developments—many different versions are available and, in some locations, the use of building paper has almost completely disappeared.

In the last few years some reservations have been made about the widespread use of housewrap and discussions of the relative merits of building paper and housewrap have appeared in the builder / remodeller press (1). Issues such as the proper installation technique, durability, and the consequences of exposure to the exterior environment during construction or over slowdowns, have still to be resolved. This paper identifies some of these issues and attempts to address some concerns about durability.

2 What is housewrap?

Housewrap is a thin, lightweight fabric that is, literally, wrapped around the outer part of the walls of the house or building involved. It is usually attached to the exterior sheathing behind the cladding as shown in Figure 1. Building paper is not housewrap and in spite of alternative terms such as homewrap or, worse, weather barrier, we shall continue to use the term housewrap as a generic descriptor.

The majority of housewraps are made of polyolefin fibers that are spun, woven, laminated or fiber reinforced to produce a lightweight fabric (2). Polyolefin is a generic term that is used to classify a number of thermoplastics such as polypropylene, polystyrene, and polyvinylchloride.

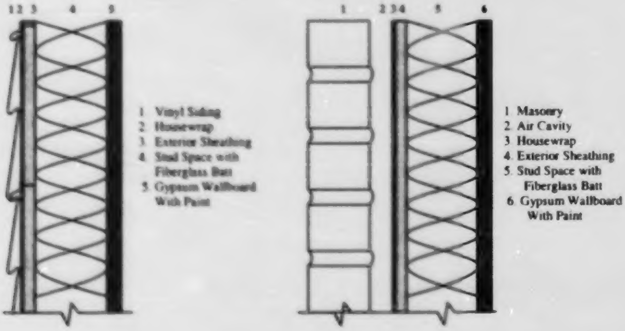

1 2 3 4 5
1 Vinyl Siding
2 Housewrap
3 Exterior Sheathing
4 Stud Space with Fiberglass Batt
5 Gypsum Wallboard With Paint
1 2 3 4 5 6
1 Masonry
2 Air Cavity
3 Housewrap
4 Exterior Sheathing
5 Stud Space with Fiberglass Batt
6 Gypsum Wallboard With Paint

Fig.1: Typical wall sections

3 Intended functions of housewrap within the building enclosure

Without using quite the same precision, the manufacturers of housewraps generally claim the following capabilities for their product:

1. A water barrier, i.e., to any wind driven rain or, for that matter, any other water that may somehow reach the outer surface of the housewrap, and then, in turn, a drainage plane for this water. The housewrap is thus an internal barrier to liquid moisture as well as an integral part of the internal water drainage system.
2. An air barrier.
3. A control layer for water vapor diffusion but to facilitate, not to retard, diffusion: the intent is to encourage the drying of the wall rather than to have the housewrap layer be a significant contributor to the retardation of water vapor flow.

Knowing the intended role of the housewrap within the building enclosure is, of course, essential to an understanding of how, when, where, and why housewrap should be installed.

4 Installation of housewrap

In wood or steel-stud framed wall systems the housewrap is generally attached to the outer face of the exterior sheathing by means of fasteners such as large head nails, nails with plastic washers, or staples. Horizontal and vertical joints are overlapped—from 2 to 6 inches depending on the manufacturer's instructions—and taped or otherwise sealed in order to stop the infiltration/exfiltration of air and to prevent the inward movement of any water. Window and door openings require proper flashing details that incorporate taping and/or sealing to prevent the transfer of both air and water. Most manufacturers provide installation guidelines that include hints intended to make installation easier.

Housewrap that is properly installed should enable the wall—and it must necessarily be a screened and drained type of wall system—to function well. Installation is, or should be, relatively simple, even when openings have to be accommodated. The most important consideration when it comes to installation is continuity: the housewrap must be properly flashed around openings, joints between overlapping layers should be taped or sealed, and any tears and holes should be taped or otherwise sealed. In addition to continuity, the type of fastener, its use and corresponding spacing must be appropriate.

4.1 Survey of housewrap use and installation

A survey was conducted at three locations in Pennsylvania in order to develop some representative data on how housewrap is actually being installed and utilized. Thirty new house construction sites were visited. During each site visit, the type of fastener, fastener spacing, joint overlap dimension, sealing method, flashing detail, and window and door opening details were documented. The data was compiled and assessed. The survey provided information that is probably also representative of installation techniques and conditions outside of Pennsylvania. These documented field conditions also provide a basis for installing housewrap for laboratory testing. Table 1 lists some of the summary findings regarding fastener type, joint treatment and window detailing.

Table 1: Installation survey

	Fastener Type			Joint Treatment		Window Detail	
	Staple	*Nail*	*Plastic Cap Nail*	*Taped*	*Not Taped*	*Taped or Sealed*	*Neither Taped nor Sealed*
Percentage	70%	27%	3%	7%	93%	27%	73%

4.2 Survey results

The results of the survey are discussed in detail elsewhere (3). The following general conclusions are however pertinent:

1. Given that the installation guidelines permit (without necessarily endorsing) the use of staples, it is not too surprising that 70% of the houses had stapled housewrap and only 3% of builders used the plastic washer-nail combination. Fastener spacing was typically irregular, but, in most instances, sufficient fasteners at appropriate spacing had been provided. The quality of attachment not the quantity of fasteners, is an issue. In the majority of the houses where staples had been installed with an automated staple gun, tears and holes in the housewrap were common. Haste and, especially, overly robust handling of the staple gun appeared to be the cause of numerous small tears and holes.
2. At locations where two separate sheets of housewrap intersected, the overlap at most of the horizontal and vertical joints was consistent with manufacturer guidelines (50mm – 150mm). However the vast majority (93%) of the housewrap joints were neither taped nor otherwise sealed.
3. Typically, the housewrap is initially applied over window openings. Then, prior to the installation of the window, an 'x' is cut in the housewrap and the resulting flaps are folded inside the rough window opening. The window is then installed. To prevent airflow and the ingress of water, the space between the housewrap and the window should be properly sealed. In 73% of the houses the window to wall joint was neither sealed nor flashed.

5 Experimental research

In a screened wall system with a vented air space behind the cladding (a pressure moderated and drained screen system), the housewrap constitutes, presumably by design, the first air-barrier layer to actually experience significant wind-induced air pressure differentials. Because of its location the housewrap is also intended to be the last, not the first, line of defense against water from precipitation reaching the inner portion of the wall. Obviously, the quality of installation will have an influence on in-service performance, but there is a need to physically investigate the likely nature of in-place performance and to better understand the contribution of the housewrap to the in-service performance of the wall system.

Water vapor diffusion or, more precisely, excessive retardation of outward water vapor flow is not a concern since the many staple or nail penetrations as well as the numerous other small holes and tears serve to reduce the resistance to vapor flow by diffusion. Air and liquid water movement can be prevented if a continuous water/air tight barrier is present.

A series of experiments for different wall construction practices was conducted to examine the resistance to air movement and to establish the characteristic behavior of the housewrap individually as well as in combination with the other components that make up the wall assembly. The results provide a good indication of the contribution of each component within the wall to the resistance of air movement.

5.1 Test specimens

The test specimens were constructed to generically mimic the best of the actual construction practices seen in the survey. Each 4 foot x 8 foot test specimen is described in Table 2. The two basic test walls are essentially the same except for the type and location of the sheathing joint. The only air leakage path permitted was at the location of the vertical and horizontal joint in the exterior sheathing and at the top and the bottom edges of the gypsum wallboard interior sheathing. The remainder of the wall assembly was sealed around the edges with caulking and weather stripping.

Table 2: Description of experimental wall assemblies

Wall #: VA & HA	
Wall Layer	**Fastener Type/Spacing (V & H)**
Vinyl Siding – Horizontal Lap	25mm (1") roofing nail – 600mm (24") spacing
Housewrap (perforated woven polyolefin) fastened to sheathing	12mm (½") staple – 600mm (16") spacing grid
12mm (7/16") OSB Sheathing on 2 x 4 studs 600mm (16") o.c. • **VA** - ***vertical joint over stud*** • **HA** - ***horizontal joint @ mid-height***	6d nail – 200mm (8") spacing on every stud
Fiberglass Batt Insulation with kraft paper in stud space	
12mm (½") gypsum wall board	37mm (1.5") screws – 600mm (16") spacing

5.2 Test procedures

Each series of wall tests was initiated by installing the wall assembly, initially only the wood framing and the OSB sheathing, into the enclosure test chamber in the BeTL facility (Building Enclosure Test Laboratory at Penn State University). Positive (push or blow) and negative (suck) air pressure testing was then conducted on the following wall sub-assemblies:

1. Exterior sheathing only attached to wood framing
2. Exterior sheathing plus housewrap
3. Sheathing plus housewrap plus batt and interior sheathing
4. Sheathing plus housewrap plus batt and interior sheathing plus vinyl siding

Each wall assembly was incrementally tested for air leakage, between a range of zero to 1000 Pa and back to zero differential pressure. The tests were conducted under positive and negative air pressures to simulate windward and leeward wind pressures. Air leakage measurements were taken with a laminar flow element after each desired pressure measurement stabilized. The test procedure was essentially consistent with ASTM E283 and the same procedure was followed in each of the eight tests in each wall series. The intent behind each series of tests was to serially demonstrate the contribution to air tightness of:

- i. the exterior sheathing, then
- ii. the addition of the housewrap and then
- iii. the addition of batt insulation and interior sheathing. Given that, in theory, a 50mm (2 inches) clear cavity is required between the brick veneer and the housewrap, this condition could also be said to be representative of masonry veneer – wood frame construction.
- iv. The addition of vinyl siding.

5.3 Results

Figure 2 through Figure 5 are the resultant air leakage characteristics that were obtained for each of the test specimens for air pressures between zero and 100 Pa. This covers much of the likely range of service-life wind pressures. To provide a relative datum, the 'stated' air leakage characteristics in the manufacturers literature is also shown in each figure. The following general conclusions may be drawn:

1. It is evident that the addition of perforated woven polyolefin housewrap does reduce the movement of air; however, it is only significant under positive pressures. The application of negative pressures to the perforated woven polyolefin results in air leakage rates that are essentially the same as the sheathing joint itself. The air tightness of the housewrap once installed is demonstrably different from that provided by testing in a laboratory and that quoted by the manufacturers of this particular housewrap.
2. Even at low pressure differentials, some degree of ballooning of the housewrap occurs under suction. Thus the air tightness of the housewrap, as well as the effectiveness of any pressure moderation, is greatly diminished.
3. In the interest of air tightness, the exterior sheathing should be installed with a vertical joint at the middle of the stud (i.e. avoid having any exposed joints either horizontal or vertical). When installed in this fashion, the sheathing is likely to be a more effective contributor to air tightness.

4. The addition of the kraft paper faced batt insulation and the interior gypsum sheathing (sealed only on the vertical edges) makes a very significant contribution to the air tightness of the wall system. With the batt insulation and gypsum board in place, the wall system reaches a level of measurable air tightness, for both positive and negative pressure differentials, that approaches recommended values for an air barrier.
5. The addition of a vented screen, here vinyl siding, served to reduce the air tightness of each wall system. The nails used to attach the siding (and similarly any cladding or screen connections) penetrate the housewrap and the exterior sheathing. In all cases, this resulted in a measurable increase in air leakage.

These quasi-static tests clearly demonstrate that for air movement control across a wall, the wall system as a whole is engaged and that, under positive or negative air pressure, the housewrap contributes in very different ways. Under negative pressures lift-off or ballooning occurs with the following possible consequences:

Capillary Break: If the air space behind the screen is intended to provide a capillary break, then this function is likely to be diminished and water can readily be transferred between the rear face of the screen and the outer face of the housewrap. Obviously, the as-built distance between the rear of the screen and the housewrap as well as the frequency and magnitude of the air pressure differential, is important.

Screen Attachment: The magnitude, nature and distribution of the forces in the lateral connectors between the screen and the rest of the wall will be very difficult to determine especially under negative wind pressures.

Durability: The housewrap is subject to many cycles of wind pressure during its lifetime. Is fatigue an issue? Is stretching an issue? Is incremental damage at the fasteners - especially with staples – an issue?

Water Barrier: The most important in-service function of any membrane attached on the outer face of the exterior sheathing is to be a barrier to external moisture, primarily rain water. How is this function affected?

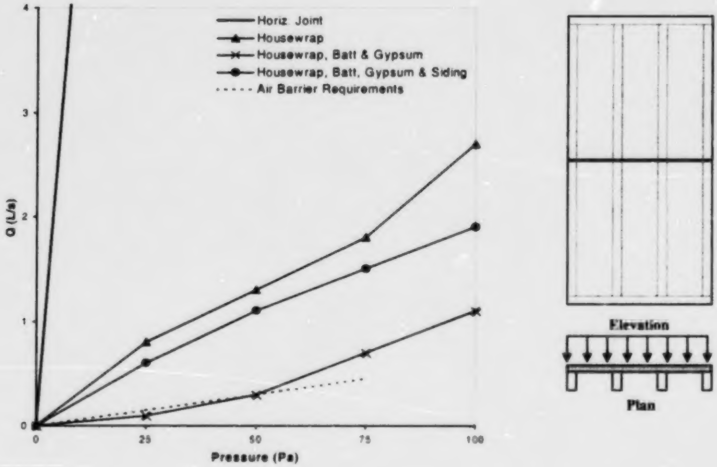


Fig. 2: Wall #: HA Subjected to positive pressure

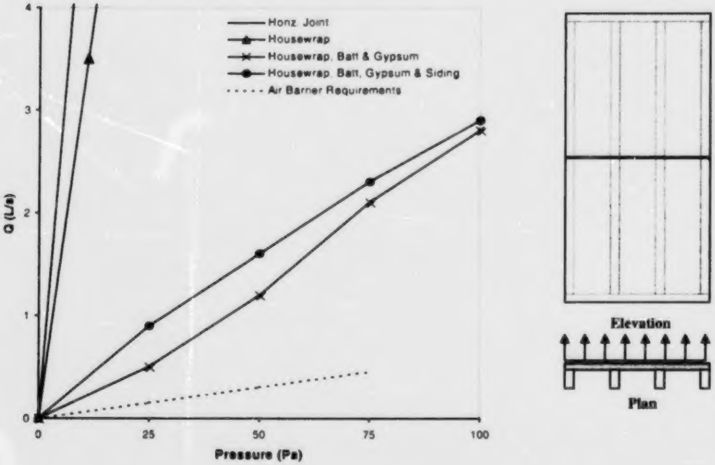


Fig. 3: Wall # HA Subject to negative pressure

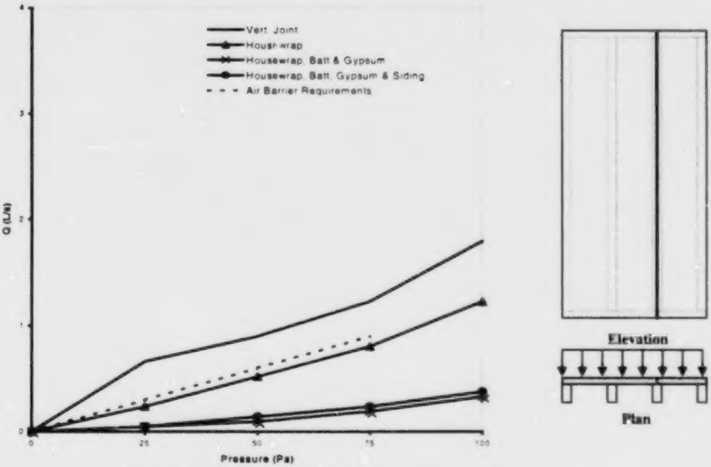


Fig. 4: Wall #: VA Subjected to positive pressure

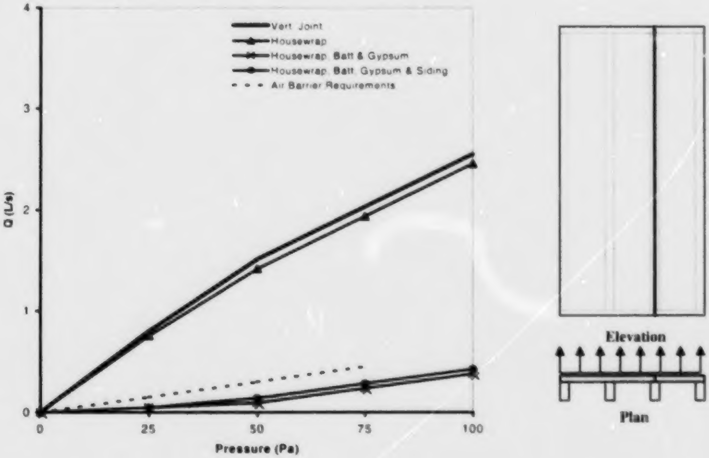


Fig. 5: Wall #: P1-VA1 Subjected to negative pressure

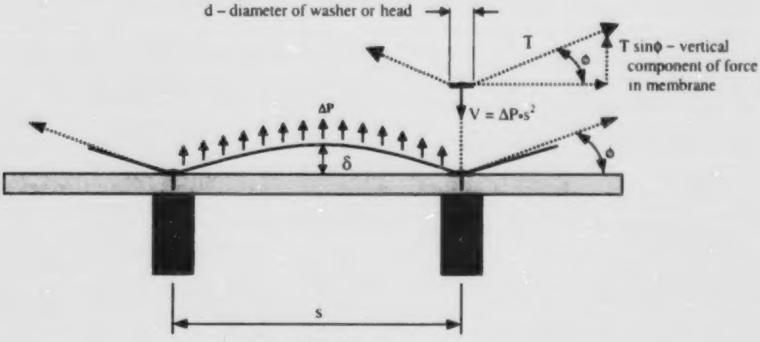
These performance issues are currently being studied but the issue of durability needs to be given particular attention. We know ballooning occurs and we know that staples are being used. The simple mechanics of what can occur under negative pressure needs to be better known. Refer to Figure 6 which illustrates this situation.

It is evident that for vertical force equilibrium

$$V \text{ (the anchor force)} = \Delta P \cdot s^2$$
$$= \text{Function of } (\sigma, t, \pi d, \sin \phi)$$

where σ, t, d, and ϕ represent the tensile stress, the thickness of the housewrap, the diameter of the washer or nail head and the tangential angle of the housewrap at the edge of the washer, respectively. It follows that:

- The larger the fastener spacing (s) the larger the anchor force (V) and the greater the ballooning (δ) for any particular housewrap material.
- The smaller the diameter of the nail head or washer, the larger the stress in the housewrap (i.e., as $d \leq$ zero, $\sigma \leq \infty$).


d – diameter of washer or head
T
ϕ
T sinϕ – vertical component of force in membrane
ΔP
$V = \Delta P \cdot s^2$
δ
ϕ
s

Square grid of fasteners
(washers shown)

Fig. 6: Ballooning under suction

- The less the lift-off (i.e., ϕ and δ) the greater the stress (σ) and hence the strain on the housewrap (i.e., as $\phi \leq$ zero, $\sin\phi \leq$ zero and $\sigma \leq \infty$). The elongation of the housewrap and the displacement (δ) are directly related to the E value or stiffness (provided tearing does not occur) of the housewrap.

This structural situation is well known; it commonly occurs across the exposed roofing membrane in a roof system with a membrane that is mechanically rather then fully adhered.

6 Conclusions

Housewrap when installed is generally assumed to provide three specific control functions, however the following reservations need to be made:

Air Barrier: While sold as an air barrier, when housewrap is installed using staples in a screened, pressure-moderated, drained wall system; it is unlikely to contribute much to the overall air tightness of the wall system. If there is no other air barrier in the wall, then under negative pressures, the contribution of the housewrap might be insignificant.

Water Vapor Retarder: As a membrane attached to the outside of the exterior sheathing, the housewrap should not impede outward water vapor diffusion. Current installation practices cannot increase the vapor resistance of the housewrap and probably do not significantly affect this functional property. We do have some reservations about the target values for permeance or vapor resistance, especially with regard to inward vapor drive under some conditions.

Water Barrier and Drainage Plane: We have yet to physically investigate these two required capabilities. How well the installed housewrap actually performs in the presence of tears, holes, folds, inadequate flashing and lack of continuity needs to be investigated.

Durability is not a property that is usually addressed in the housewrap literature except for some general notes regarding ultraviolet exposure. We have very real concerns about the naturally cyclic and reversible nature of the wind and the resulting air pressure differentials that the housewrap must accommodate. At this point additional research is needed. In particular the water barrier capabilities of housewrap need to be properly investigated.

7 References

Fisette, Paul (1998) "Housewrap vs. Felt", *Journal of Light Construction*, v 17, No. 1, pg. 48.

Greenlaw, Bruce (1994) "Sizing up Housewraps", *Fine Homebuilding*, pg. 43.

Bosack, Ed and E.F.P. Burnett (1998) "The Use of Housewrap in Walls: Installation, Performance and Implications", Pennsylvania Housing Research Center, Report # 59.

90 A DECISION SUPPORT SYSTEM FOR PRELIMINARY DESIGN

E.H. de Groot, S.M. Mallory-Hill, R.H.M. van Zutphen, and B. de Vries

Eindhoven University of Technology, Eindhoven, The Netherlands

Abstract

The design of buildings is a complex task for a variety of reasons. In the conceptual stage, particularly in the inception phase, a small number of people make decisions that have far reaching impact on the final result in terms of efficiency and effectiveness. Decision-making in the inception phase of design relies on available information which may be incomplete, (e.g. maintenance costs), or overly complex (e.g. code requirements). Furthermore, in the near future the consideration of life-cycle issues will be essential if not compulsory in all construction projects. Therefore, a smart, interactive, and easy-to-use system that provides a way of viewing the potential consequences of their design decisions would be useful to all participants involved in early design. This paper presents results from ongoing research at the Eindhoven University of Technology concerning the development of such a Decision Support System (DSS) for the design of indoor environments for offices. Items addressed include the utilisation of existing databases and expert knowledge about the entire building life-cycle.

Keywords: Building Environmental Engineering, Performance-Based Design, Design Decision Support Systems

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1 Introduction

The Building Physics group is a section of the Faculty of Architecture at the University of Technology in Eindhoven [TUE]. One of the units within the Building Physics group is the Indoor Environment Unit, which investigates, amongst other things, the comfort of human beings inside buildings. This includes: thermal comfort, acoustical comfort, visual comfort, and air quality.

The study of *strategic building performance planning and evaluation* is one of the unit's research programs. Strategic building is based on the philosophy of providing a good match between client needs and building performance over time. This project was initiated to examine how knowledge gained from the evaluation and testing of the physical aspects of office building designs can be used to improve the long-term performance of new buildings. The research program focuses on the early design phase because it is felt that there is a particular need for support in this area (Hill 1997) and (Groot 1998).

One of the main activities of this project is the development of Information & Communication Technology [ICT] that can be used to provide multiple-level feedback about a proposed or existing design. This research project is being done in collaboration with the Design Systems group, who specializes in the development of computer-based design aids. The project has the following objectives:

1. Improve communication between members of a design team,
2. Improve knowledge transfer from research institutes to building practice,
3. Improve knowledge transfer from the occupation phase to the design phase,
4. Support a strategic performance-based design approach

1.1 Improve communication

All participants involved in early design need to be able to understand the consequences of their design decisions on other participants' field of work. For example, let us say that the architect wants to make the building look transparent and decides to increase the window area in the façade. This may increase the cooling load in summer and therefore the building service engineer should be aware of this change in the design, because there may be a need to increase the capacity of the cooling system. A computer-based design decision support environment should include warning systems to flag when conflicts in design goals occur.

1.2 Improve knowledge transfer

Very often people working in the building industry are not aware of new innovations found through research done at universities and research institutes. Likewise, people working in research institutes may not be aware of the issues related to the application of innovation in practice. Therefore, the design decision support environment should provide a link between researchers and practitioners. The system should contain up-to-date knowledge and act as a repository for past experience that can be readily accessible to practitioners in their design environment.

1.3 Improve knowledge transfer between life-cycle phases

During a building's life-cycle a multitude of decisions are made that impact on performance. Very often people involved in the different phases of the building life-cycle can have problems keeping track of decisions, particularly if they were not involved in the decision-making process themselves. As a result, designers may be unaware of how a design decision impacted on issues that only become apparent after the building was constructed (such as durability or maintenance). A decision support environment should provide the possibility to store decisions and the reasons why they are made. Moreover, it is important to store how previous decisions relate to those we may make in the future. This is helpful for maintaining user objectives throughout the design cycle, but also makes it possible to learn from the knowledge acquired during a previous project.

1.4 Support a strategic, performance-based approach

Strategic building performance planning and evaluation, as described in (Hill 1997), is a combination of user requirement analysis techniques from strategic facilities planning with performance-based representation and evaluation of design. In this approach, total building value is evaluated on the basis of a building's fit with its human environment (from individual occupants to the global community) at any given time in its life-cycle (see Figure 1).

The advantage of using such an approach is that it concentrates on how well a design is satisfying human needs, rather than on a particular design itself. In terms of design, this means that as a designer or a contractor, we do not have to use a certain prescribed approach if a new or different one is able to meet the needs of our client. The disadvantage is that it requires a great deal of multi-disciplinary knowledge to be able to make design decisions that satisfy the performance of systems on multiple levels. This is why a design decision support environment is needed that can help measure and evaluate the performance of proposed design solutions across various levels.

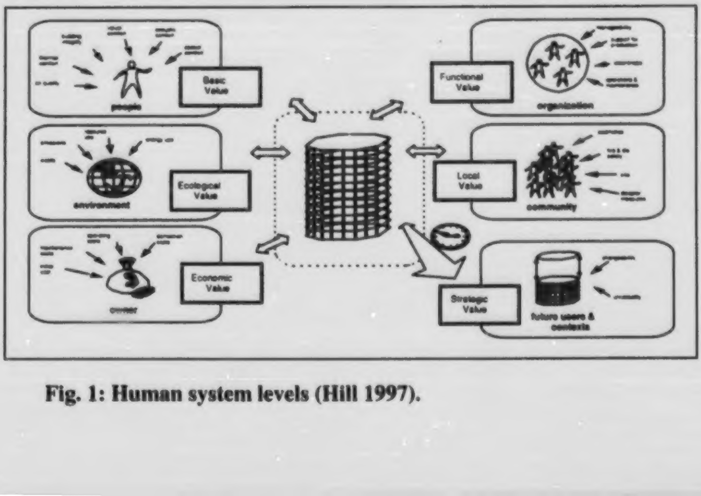

Fig. 1: Human system levels (Hill 1997).

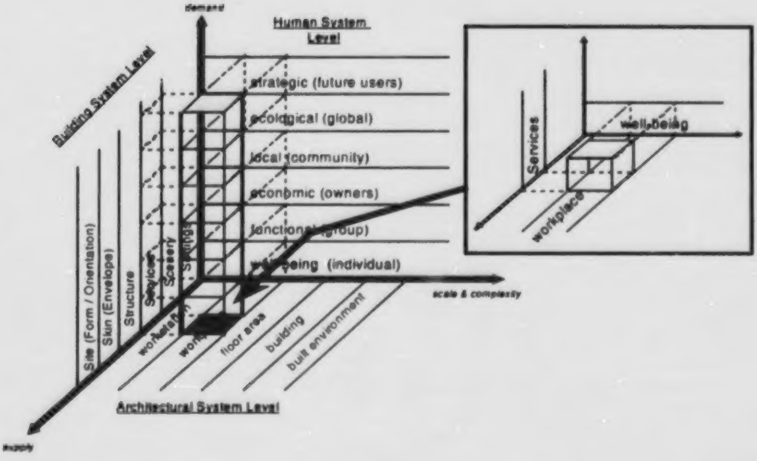
demand
Human System Level
Building System Level
strategic (future users)
ecological (global)
local (community)
economic (owners)
functional (group)
well-being (individual)
scale & complexity
Site (Form / Orientation)
Skin (Envelope)
Structure
Services
Scenery
Stuff
workstation
workplace
floor area
building
built environment
Architectural System Level
supply
Services
well-being
workplace

Fig. 2: Office building performance evaluation (Hill 1997)

2 Performance-based evaluation

To better understand how many levels of performance one must be aware of, (Hill 1997) uses a 3D matrix model to describe the nature of performance evaluation of office buildings (Figure 2):

1. *Architectural system levels*: the scale at which the problem is evaluated. Beginning with the workstation and moving towards consideration of the entire built environment.
2. *Building system levels*: the supplied performance of the various building systems (after the "6-S's of (Brand 1994)).
3. *Human system levels*: the demands the various stakeholders of the building have from individual occupant requirements (well-being) to those of the global community (ecological) and future users (strategic).

Similar to (Wise 1990) and (Preiser et al. 1988), these matrices are used by advocates of performance-based approach to identify what elements of performance that are "measured, evaluated and used to improve buildings"(ibid 39). By breaking down each level in the matrix further into sub-systems (and sub sub-systems) Hill reveals the immense scope and complexity of evaluations possible (some of the breakdowns for human systems are shown in Figure 1). Currently, researchers at the TUE use the model to identify a focus for their own evaluation research (a particular "cube") in relation to the whole.

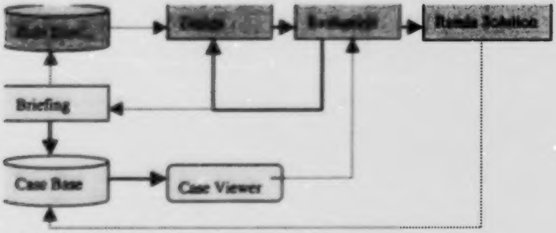

Fig. 3: Simplified model of the DSS

3 Building evaluation program research projects

The long term goal of the Building Evaluation Program is to develop a Design Support System Environment. A schematic model of the proposed system is shown in Figure 3. The square boxes identify the four main processes to be supported by the system: *Briefing*, *Design*, *Evaluation*, and *Retain solution*. At the present time several projects are underway to develop aspects of this proposed system.

Two of the studies investigate how knowledge-based systems can provide support during the early stages of design, but they use different strategies. The first by providing feedback from evaluation of design concepts proposed by the user and the second by providing feedback from the stored evaluations of existing buildings or *cases*. Other related studies are concerned with the development of design interfaces using *Virtual Reality* and the automatic generation of design knowledge using *Agents*. The following sections provide more details about each of these projects.

3.1 Case based design for office environments.

This research project began in 1996. One of the objectives of the research was to learn more about in-use studies of workplace environments to feed forward technical knowledge into the early design stages. As part of this work a study was undertaken to collect performance data about office rooms located at TUE in the Netherlands (Zonneveldt and Mallory-Hill 1998). The office rooms are more or less identical except that they each contain a different innovative lighting system. This work has formed the basis for the development of a World Wide Web-based Case-based Design Assistant [CBDA] to support the conceptual design of workplace environments, see also (Mallory-Hill 1998).

The CBDA works by retrieving, via the WWW, the evaluated office lighting designs. The interface between the designer and the system is through the user requirements. The designer uses the system to describe their project's characteristics, such as location, façade orientation, building type, and space type. After that the user must decide on a "Design goal". For example: an energy efficient artificial lighting design, or a minimum cooling load. Further, the general items of the building brief, such as amount of persons in the room, work tasks and importance, average age of the persons, are also entered. These building brief items are translated by the system into sets of technical requirements (demands) expressed in ranges, such as illuminance level (200 -500 lux), temperature (20 -24 degrees Celcius).

The system uses sets of technical requirements to search the Case Base for relevant cases. Sets are used because the quality of work environments in-use are measured in various ways and therefore the cases contain more than one description of their performance. Therefore the designer must decide on one of four types of performance evaluation criteria for the system to use to retrieve the case:

- Designed: measured by testing a model of the final design (e.g. simulation).
- Observed: measured by an expert or panel of experts.
- Perceived: measured by the users or occupants of a particular environment.
- Measured: captured through monitoring of physical phenomena.

Cases that are retrieved from the case-base are those that that match the demand criteria (designed, observed, perceive, or measured). These are shown to the user in a Virtual Reality-view and in a plan view. The technical performances are shown in a graph, making comparison of retrieved cases possible.

3.2 Multi-criterion analysis for the design of office lighting

This research study involves the development of a DSS for evaluating energy efficient office lighting designs using a multi-criterion analysis approach. This project began in November 1996. The study focuses on:

- Two building systems: Daylight and Artificial lighting.
- Five Performance Requirements: Energy efficiency, Visual comfort, Initial costs, Maintenance, and Installation aspects.
- One architectural level: Workplace in an office building.

The first goal of this project is to capture and model knowledge from the field of office lighting design in order to answer the following questions: How do lighting experts pick a daylight-system and an artificial lighting system based on the requirements set in a brief? Will these two lighting systems be integrated, and if yes, how is this done? How do experts rate the performances of the various solutions, compared to one another?

The second goal is to develop, implement and test a prototype DSS for office lighting designs. This prototype consists of a design and evaluation environment supported with captured knowledge stored in a *Rule Base*. The following is a brief description of how the system works (see also Figure 3):

Rule Base: The items of the general brief, determined in the Brief module, that are relevant for lighting are copied into the DSS (such as location, façade orientation, building type, space type). The user must decide on some additional requirements especially for lighting design (such as appearance of the room, colour of light, importance of glare control and view). The Rule Base contains rules that indicate which design solutions can be chosen to meet these requirements. An example of a rule can be: IF "location" = "NL" THEN "Windows type.Daylight system" = "Air exhaust" OR "Double glazing".

Design: The DSS shows a default solution that the user can change the features of:

- The daylight system (e.g. window type, additional elements and architectural features),
- The artificial system (e.g. lighting distribution, light source and luminaire type),
- The control (position, switch, and automation), and
- The maintenance aspects (cleaning and relamping).

Options that meet the requirements set out in the project brief are distinguished from the options that do not.

Evaluation: Five different performances of the design or case, (visual comfort, energy efficiency, initial costs, installation aspects, and maintenance) are measured and shown in a radar chart. After this an analysis of the results will take place by using:

1. Rules of thumb: is one or more of the performances unusual compared with common practice?
2. Sensitivity analysis: the tool automatically adjusts unlocked parameters to improve one of the performances so that it more closely matches the target set by the user. The tool indicates which change in which parameter has the best result.

The first prototype DSS was developed under the framework of the EU-sponsored project: Energy Impact Knowledge-based System [EIKS] in 1997. At that time workshops were organized with Dutch building experts, as the future users of the DSS, to define the specifications for the system. A demo DSS was developed and tested with focus groups of building experts. The results of these and the earlier workshops can be found in (De Groot and Pernot 1998). In 1998 additional research, including an internship at Lawrence Berkeley National Laboratories, helped generate new ideas to improve the prototype further. The proposed changes include:

1. Rules must be implemented to assist the user translating the requirements into design solutions. Rules are also needed to indicate which artificial lighting system can be applied with the chosen daylight system.
2. A virtual prototype of the workplace should be developed, where the user can see the changes to the solution he or she has made.
3. The relative performances for each component should be checked and adapted with some lighting experts to get realistic averages. The performance of the total solution should be shown in radar charts on which the user can also indicate which performance scores are required

3.3 Virtual reality-design information system

One of the aims of the Virtual Reality-Design Information System [VR-DIS] program of the Design Systems group is the development of 3D design systems for conceptual design using Virtual Reality technology (Vries and Jessurun 1998). Currently much of the research effort is being spent on creating the underlying design information data structure and on the development of an intuitive user interface for 3D-shape manipulation. So far three approaches to making design knowledge have been explored (divided according to purpose and interaction style):

1. *Educational;* a stand-alone system for organizing architectural design examples of special interest. The system is a WWW browsing system and supports limited search facilities. In this approach there is no data sharing between the DSS and the design system.
2. *Consulting*; a system that actively supports the designer in the search for a solution to a specific design problem. Multi media such as 2D/3D graphics and hypertext is used to show all information about a specific solution and to give context about the designers intent. The selected solution can be copied into the current design. This approach requires that the data structure of the selected solution can be transferred to the design environment. In the VR-DIS environment this means that building elements (e.g. shades, luminaire type) and their characteristics (e.g. position, colour) are modelled as "Features" (Achten and Leeuwen 1998). In this approach there is no real interaction between the design environment and a DSS.
3. *Expert agent*; from within the design environment the designer requests support on a specific subject. Possible solutions are presented and can be incorporated into the current design. The effects of the selected solution (e.g. lighting, shading) are simulated in the design model. Changes in the design model will directly cause the system to respond about the performances. Performances are superimposed upon the building element geometry (e.g. light distribution).

The consulting approach is chosen to be used for the Building Evaluation Program since it allows for a stand alone system, while at the same time the outcome can be used in the VR-DIS design environment. The same system framework can be used to develop DSS's for other domains such as structural engineering, building management. The major drawback is that solutions brought into and then modified in the VR-DIS environment cannot be re-entered back into the DSS.

3.4 Multi-agent systems

Design systems should support the ability to communicate, collaborate and co-ordinate design decisions with clients, users, legislators, advisers, engineers and contractors, deal with conflicts, and negotiate a consensus. It is hard to have this kind of interactive design system if the expert knowledge within the design system, as in the current projects of the Building Evaluation Program, is predefined during the development of the system. A possible solution to this problem could be the development a system with learning capabilities so that knowledge could also be attained during operation.

The above issues are the main topics in the research about multi-agent systems. Multi-agent systems model and support users in coordinating and performing complex co-operative tasks - the same kinds of tasks one can observe in architectural design projects. To research the possibilities of an agent-based design system the DESSYS research project (Veghel 1998) was defined. The DESSYS research on multi-agent systems is particularly focused on co-operation and collaboration, team- and coalition formation, information-sharing among the design team, and joint beliefs, goals and plans.

The most important asset of an agent is its ability to learn through its interaction with system users. By learning, a user interface agent can accumulate so much knowledge, that it becomes to some extent, an equivalent of the person it is attached to. If this occurs, we state that the user interface agent has become a knowledge-based agent. The extent to which this will happen is dependent on the learning and storing capabilities of the agents currently being developed.

4 Conclusions

In order to increase the service life and durability of buildings we need to ensure a good fit between user requirements and building performance over time. The Building Evaluation Program at the TUE examines how to improve the long-term performance of new buildings by feeding back knowledge gained from the evaluation and testing of existing office buildings. The benefit of learning from the mistakes of the past is obvious. However, the practical use of this knowledge remains uncertain.

During design, particularly in the early stages, it is important to have the means of establishing clear requirements and anticipating the performance of proposed solutions. In practice there are many barriers to getting the kind of information and technical feedback needed to be able to do this. We believe it is possible to improve access to this knowledge through the use of information and communication technology [ICT]. In this paper we have described the current research being undertaken at the TUE to develop ICT systems that are capable of providing design decision support:

- *Case Base-based Reasoning* is being used to provide a link between post-occupancy evaluation and user requirements of new office design projects.
- *Multi-criterion analysis* is being used to access the domain knowledge of lighting engineers and other experts for the evaluation of lighting system concepts.
- *Virtual Reality* provides us with possibilities for improving the communication between team members by providing a shared and easily understandable interface.
- *Multi-Agent systems* are being investigated to provide us with the possibility of learning from design information already stored in the computer.

Of course, the development of these systems is still at an early stage. A great deal of effort is required to assemble the information and expertise used by the systems. Future plans include striving for a greater integration of the support system environments and testing with user groups of building design professionals.

5 References

Achten, H.H., and J.P. van Leeuwen (1998) A Feature-Based Description Technique for Design Processes: A Case Study, on CD-Rom: *Proceedings of the 4th Conference on Design and Decision Support Systems in Architecture and Urban Planning*, (ed. H.J.P. Timmermans), 26-29 July 1998, Maastricht..

Brand, S. (1994) *How Buildings learn; What happens after they're built.* Viking Penguin: New York.

Groot, E.H. de (1998) *Definitions for the pilot Decision Support System for Design and Evaluation of Integrated Daylight and Artificial Lighting systems.* Internal report, Eindhoven University of Technology, Eindhoven.

Groot, E.H. de and C.E.E. Pernot, Energy Impact Knowledge-based System project: Outcomes of the Netherlands Workshops, on CD-Rom: *Proceedings of the 4th Conference on Design and Decision Support Systems in Architecture and Urban Planning*, (ed. H.J.P. Timmermans), 26-29 July 1998, Maastricht.

Hill, S.M. (1997) *Intelligent Tools for Strategic Performance Evaluation of Office Buildings.* Internal report, Eindhoven University of Technology, Eindhoven.

Mallory-Hill, S.M. (1998) Building a Case-Based Design Assistant for Workplace Environment Design, on CD-Rom: *Proceedings of the 4th Conference on Design and Decision Support Systems in Architecture and Urban Planning*, (ed. H.J.P. Timmermans), 26-29 July 1998, Maastricht.

Preiser, F.E., Rabinowitz, H.Z., White, E.T. (1988) *Post-Occupancy Evaluation.* Van Nostrand Reinhold Company, New York.

Veghel, M.M.P.H.L. van and Zutphen R.H.M. van and Timmermans, H.J.P. (1998) Agent-Based Design Information Systems: An Introduction: *Proceedings of the 4th Conference on Design and Decision Support Systems in Architecture and Urban Planning*, (ed. H.J.P. Timmermans), 26-29 July 1998, Maastricht.

Vries, B. de and A.J. Jessurun (1998) An Experimental Design System for the very Early Design Stage, on CD-Rom: *Proceedings of the 4th Conference on Design and Decision Support Systems in Architecture and Urban Planning*, (ed. H.J.P. Timmermans), 26-29 July 1998, Maastricht.

Wise, J.A. (1990) Control is beautiful: Measuring Facility Performance as if People (and Buildings) Really Mattered, in *Performance of Buildings and Serviceability of Facilities*, (eds. G. Davis and F.T. Ventre). American Society for Testing and Materials: Philadelphia, pp 121-141.

Zonneveldt, L. and S.M. Mallory-Hill (1998) Evaluation of Daylight Responsive Lighting Control Systems, in *Daylighting '98; Conference Proceedings*, (ed. M.R. Atif) 11-13 May 1998, Ottawa,: pp. 223-230.

91 EVALUATION OF THE LONG-TERM PERFORMANCE OF WATER REPELLANTS ON RENDERED AUTOCLAVED AERATED CONCRETE

H. Kus and P. Norberg

Royal Institute of Technology, Gavle, Sweden

Abstract

External wall surfaces made of porous materials are known to be susceptible to the effects of wind-driven rain. Water repellants have become widely used to mitigate the apparent problems arising from rain deposition on such wall types. Although manufacturers claim that water repellants perform adequately for extended periods of time, there are still apparent premature failures that occur within a few years of application. To date, various kinds of water repellants have been developed but limited research has been carried out particularly on the long-term field exposure testing from which the real effect of climate on the properties of water repellants can be assessed. This study aims to investigate the long-term performance and durability of water repellants on rendered autoclaved aerated concrete (AAC) by physical and chemical analysis of fresh, naturally and artificially weathered samples. Six sample blocks of AAC were used; one uncoated and untreated as control block and five rendered blocks, one of which was untreated, two treated with silicone, one painted with a styrene-acrylate paint, and one covered with expanded ePTFE membrane employed as the ideal coating. A preliminary test was done for selection of the paint for sealing the sides of the samples against water and vapour ingress. Continuous monitoring of moisture contents and temperatures of naturally exposed samples is also included in the study. WETCORR sensors and nail electrodes are used to measure surface and bulk moisture contents, respectively. This paper focuses on the preparation of test blocks and moisture monitoring and presents the first results derived from field measurements. The results indicate that the experimental set-up works as planned and it is expected that an assessment of the service life of water repellants based on moisture monitoring will be possible after the samples are exposed to longer periods.

Keywords: Autoclaved aerated concrete, field exposure, long-term performance, moisture monitoring, water repellants, WETCORR

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1 Introduction

Water repellants are increasingly used for protection of porous mineral building materials subjected to prolonged wetting by driving rain. Although their performance has been confirmed by research, there are still questions on their long-term durability and service life. Existing research on performance of surface treatments is mostly focused on concrete structures and the protection of historical buildings built of stone, brick and wood, and is based primarily on short-term laboratory testing.

In order to evaluate the long-term performance of water-repellent treatments on porous mineral building materials, a long-term field exposure and a short-term accelerated test programme have been initiated at the Royal Institute of Technology (Sweden), Centre for Built Environment. The research work includes continuous monitoring of moisture, which is considered as the major factor in determining durability of porous building materials, and physical and chemical analysis of fresh, naturally and artificially aged samples.

For the surface finishing coatings, AAC and mineral rendering were chosen as substrates being representative of porous building materials. AAC has an extensive use both in residential and industrial construction due to its thermal properties, that lead to energy savings by lowering heating costs and indirectly preventing air pollution. In order to get higher performance, AAC walls are rendered mostly with lime-cement mineral rendering.

Sample specimens used in this work include water-repellent treated and painted, rendered AAC blocks, in addition to reference blocks. Test blocks measuring 400 mm × 300 mm × 150 mm were used for continuous monitoring of moisture and temperature. To investigate the real effect of atmospheric agents on water-repellent properties and to observe the degradation process, an additional group of test samples measuring 130 mm × 130 mm × 150 mm have also been naturally exposed. These specimens will subsequently be used in a laboratory analysis at specific intervals. Short-term accelerated ageing tests will involve exposure to UV (and water) followed by freeze-thaw cycling. The analysis of microstructure and the surface treatments will be undertaken using microscopic (thin sections and SEM) and spectroscopic methods (FTIR).

The present work focuses on the preparation of test blocks and monitoring of surface and bulk moisture contents of the test samples.

2 Durability of rendered AAC walls with and without water repellants

Performance of external walls is greatly affected by weathering elements, thus durability and service life of the wall component vary according to the severity of exposure. The use of surface finishing coatings mainly aims to minimise and delay the deteriorating effects of the atmosphere, maintaining an aesthetically pleasing appearance. By sacrificing themselves, surface coatings extend the life of the substrate and therefore play a major role in the long-term performance and service life of the entire wall system.

Different surface coatings require different maintenance and repair periods depending on their properties and effects on the facade. Indeed, the service life of the external walls depends directly on the extent to which those intervals of maintenance are kept.

Due to being directly in contact with atmospheric agents, such as rain, solar radiation, temperature and wind, surface coatings may fail prematurely. This in turn will lead to deterioration and damages in the wall structure unless remedial action is taken. Among the atmospheric agents, moisture plays the major role in premature deterioration (Kus and Kalmar 1998). Thus the moisture storage of a wall can be a potential indicator of long-term performance for a given construction type and location. The amount and duration of moisture in the wall structure can be regarded as the two most important characteristics for assessing the long-term effects.

Field inspections on rendered AAC walls show that organic external renderings degrade faster than inorganic systems (Nygren 1998). Being vapour-tight to some extent and greatly affected by UV radiation and thermal variations, organic external coatings degrade relatively fast compared with inorganic ones. During the desorption period, the transport rate outwards of water vapour is higher in an inorganic system. It takes a longer time for organic coatings to dry out, due to their lower vapour permeability. Therefore, organic coatings may contribute to insufficient drying of the wall which can potentially lead to increased deterioration. The adhesion of the coating to the substrate is also weakened by excessive moisture, which accelerates the ageing process. Another reason for the observed differences between these coatings is that the organic external renderings inspected were not maintained as frequently as prescribed.

On the other hand inorganic external renderings treated with water repellants perform much better owing to their hydrophobic effects (Kus and Kalmar 1998). By reducing the ingress of water, water repellants are expected to minimise and delay the degradation mechanisms related to rainwater. As they are highly protected from the UV radiation by penetrating the substrate, a long durability is achieved compared to other surface coatings. The effectiveness and durability are affected greatly by the properties of the substrate, such as surface porosity and moisture content, and the amount and the uniform permeation of the water-repellent material applied (McGettigan 1992).

3 Experimental

3.1 Test specimens

Six sample blocks measuring 400 mm × 300 mm × 150 mm were used for continuous monitoring of moisture and temperature, including uncoated and untreated AAC as the control sample block. AAC blocks, 150 mm thick and density 450 kg/m^3, were employed as substrate to the coating systems tested. Four samples were coated with three layers of lime-cement rendering for a total thickness of 12 mm, one of which was untreated, one treated with a water repellant at the surface, one coated with styrene-acrylate paint, and one covered with expanded ePTFE membrane (GORE-TEX®) employed as an ideal top coating. The sixth sample was coated with three layers of lime-cement rendering containing water-repellent additive, having a total thickness of 12 mm. The AAC blocks were obtained from Yxhult AB. The renderings were obtained from Optiroc AB and Yxhult AB. The representative samples of water repellants in powder and liquid form were obtained from Wacker-Chemie. The styrene-acrylate paint was selected among the paints commonly used for mineral substrates in order to compare the effectiveness of water repellants and was obtained from Snöland Fasad AB. In Table 1, the coating systems are summarised according to their corresponding codes.

The side faces of the sample blocks were then sealed with four layers of epoxy paint, except the surface to be exposed. Consequently moisture absorption and desorption through the coatings could be controlled.

Table 1: Test blocks

Code	Substrate	Render	Top coating
O	AAC	-	-
B1	AAC	Lime-cement	Silane-siloxane emulsion
B2	AAC	Lime-cement	-
C2	AAC	Lime-cement	(Expanded) ePTFE membrane
C3	AAC	Lime-cement	Styrene-acrylate paint
G2	AAC	Lime-cement with silicone resin additive	-

3.2 Pretest for selection of the tightening material

A preliminary test was first carried out in order to obtain the best material for sealing the sides of the test blocks, which should also be durable to outdoor conditions during the whole exposure period. AAC test pieces measuring 50 mm × 50 mm × 75 mm were completely coated with seven different types of waterproof coatings with 3 - 5 layers by painting and immersion. The coatings were stearine, aluminium primer, silicone rubber and alkyd, epoxy, polyurethane and ePTFE paints. They were left to dry completely for several days before the tests. Water and vapour ingress were tested. The samples were first put in the climate cabinet at 95%RH and 20°C, and weighed periodically during 30 days. Afterwards, they were completely immersed in water and weighed periodically during 10 days. Epoxy paint proved to be the best among the conventional coatings used in this experiment. The results from this study are given in Fig.1 and Fig.2.

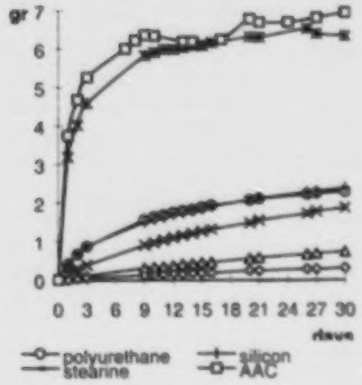

Fig. 1 : Vapour uptake

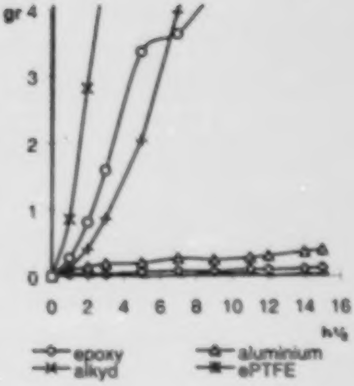

Fig. 2 : Water uptake

3.3 Moisture and temperature measurement set-up

3.3.1 Moisture sensors

WETCORR sensors (Norberg 1993) are used for measuring time of wetness (TOW) at the exposed surfaces, while epoxy coated nail electrode pairs are used for measuring moisture content inside the blocks.

Two WETCORR sensors were mounted on the exposed surface of two blocks, by gluing with epoxy resin. Nail electrodes (26 pairs) were driven into five different depths from the back of the blocks. The moisture sensors were arranged as shown in Table 2. The nail electrode pairs were placed approximately 12 mm apart. The joints at the back surface were then sealed with epoxy resin so that any leakage was prevented. The connections of the nails to the cables leading to recording devices were also sealed with silicone.

3.3.2 Thermocouples

To measure the temperature, copper-constantan-type thermocouples were used. Two thermocouples were mounted on the exposed surface of the blocks with WETCORR sensors, positioned approximately 50 mm beside the sensor and 55 mm from the bottom. The wire at the surface was glued with epoxy resin and painted white, the same colour as all block surfaces, in order to avoid any inaccurate measurement due to different radiation heat exchange. Twelve thermocouples were inserted into the blocks at the same depths as the moisture sensors, from the back (Table 2). The joints at the back surfaces were sealed. In addition to surface and inner temperatures, air temperature was also measured by one thermocouple kept in a self-ventilated radiation shield.

Table 2: The arrangement of sensors and thermocouples in test blocks, distances from the back of AAC (thickness of AAC blocks 150 mm and rendering 12 mm)

Sample	Surface	160 mm	156 mm	152 mm	148 mm	123 mm	78 mm
O					O-4 O-4T	O-5 O-5T	O-6 O-6T
B1	B1-W B1-T	B1-1 B1-1T	B1-2	B1-3	B1-4	B1-5	
B2	B2-W B2-T	B2-1 B2-1T	B2-2 B2-2T	B2-3 B2-3T	B2-4 B2-4T	B2-5 B2-5T	
C2		C2-1 C2-1T	C2-2	C2-3			
C3		C3-1 C3-1T	C3-2	C3-3	C3-4	C3-5	
G2		G2-1 G2-1T	G2-2	G2-3	G2-4	G2-5	

W : WETCORR sensor, T : Thermocouple

3.3.3 Multiplexer-datalogger

The moisture sensors and thermocouples were connected to the terminals of two multiplexers controlled by a datalogger from Campbell Scientific Inc. The sensors are scanned at five-minute intervals and the averages of resistance and temperature are stored every hour. For convenience and safety reasons the data are automatically retrieved every day and stored on a VAX computer for further processing on a PC.

The recording devices were placed in an insulated box under the test rack with a box temperature regulated to 25°C by miniature heaters. In this way the equipment is kept in a stationary environment independent of atmospheric conditions.

3.4 Orientation and exposure period

Test blocks are exposed on a 45° metal rack facing south, on the roof of the KTH-Built Environment building in Gävle (Fig. 3). In this case, the field study may be considered as a naturally accelerated ageing test.

The weather conditions at the exposure area have been monitored in close proximity to the test rack and the yearly averages according to data from 1997 are as follows:

temperature, cold season	: max. 16,9 °C min. - 15,6 °C (October-March)
temperature, warm season	: max. 29,6 °C min. - 4,8 °C (April-September)
relative humidity	: 74 %
total rainfall	: 605 mm
wind direction and speed	: (44 %) S – 2,6 m/s
UV-radiation	: 138 MJ/m^2/year

The exposure programme started on 25 September 1998 and will run for at least 18 months in order to subject samples to a wide range of weather conditions.

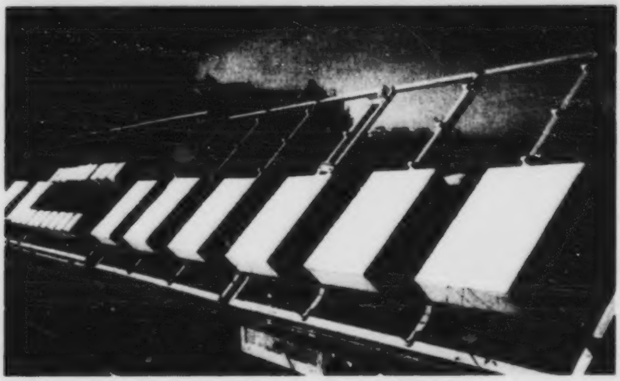

Fig. 3 : Test samples on the rack

3.5 Presentation of data

Temperatures in °C and the logarithmic values of resistances between 1-10000 k• are presented in Fig. 4 and Fig. 5 below.

4 Results and discussion

The results obtained after one month show that sample B2, the untreated rendered AAC block, has the highest level of moisture content. Moisture reaches even to the deepest point measured, which is approximately 40 mm from the exposed surface. The moisture fluctuation near the surface is, as might be expected, greater than at larger depths. Fig. 4 presents the measurement results from sample B2 for a selected period.

Sample G2, the block with water repellant as additive, shows almost no capillary absorption whereas sample B1, the block with water repellant as surface treatment, displays some capillary absorption near the surface area. The amount of moisture for B1-1, is very small and constant, which may be a result of inaccurate measurement. The precise reason for this anomaly is not known at the time of writing. Measurement results from sample B1 for a selected period are presented in Fig. 5.

Sample C2, the ideal block that was covered with ePTFE membrane and, as expected, has not absorbed any water at all.

The control sample O, the untreated and uncoated AAC block, has a capillary absorption reaching almost to the middle of the block. However, the moisture content level seems to be less than the one in rendered AAC, i.e. B2. This can be explained by the pore structure of the rendering system and the AAC substrate, respectively.

Sample C3, the styrene-acrylate painted block, has a capillary absorption only reaching to some parts of the rendering. The AAC substrate is almost dry.

Since the calibration of measurements is not yet completed, the results can only be compared for similar samples at specific depths. The absolute moisture contents will be calculated after calibration of each pair of electrodes.

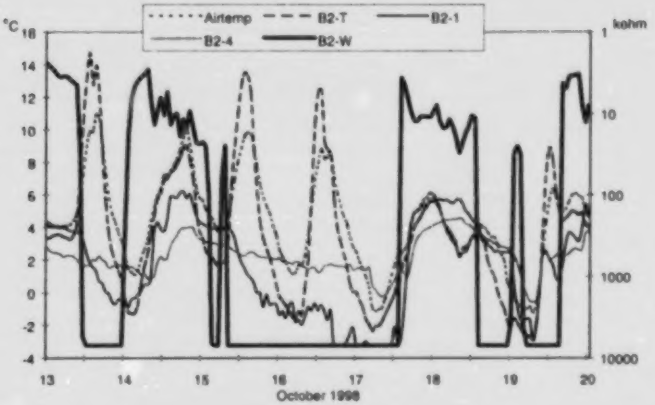

Fig. 4 : Air temperature and surface temperature, surface and bulk moisture measurement results of sample B2 for a selected period

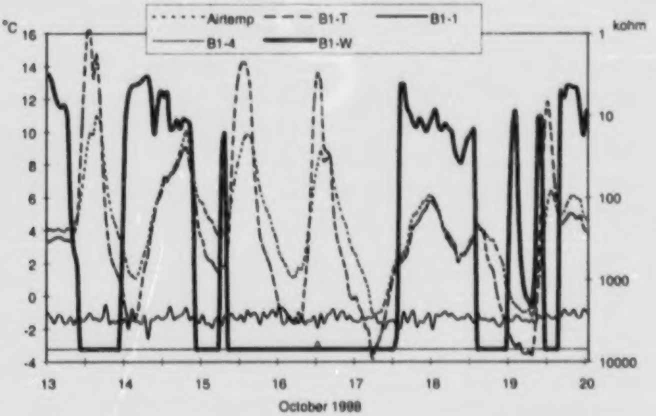

Fig. 5 : Air temperature and surface temperature, surface and bulk moisture measurement results of sample B1 for a selected period

5 Conclusion

The results obtained so far show that the field measurement set-up is working properly. However, it is too early to draw any conclusion for the assessment of the service life of water repellants based on the moisture measurements, whereas this should be possible to accomplish after a longer exposure time. It will also be likely to obtain information on the effects of water repellants on the freeze-thaw deterioration of the wall samples.

In this research work

- moisture characteristics,
- long-term performance and service life evaluation and
- degradation processes and ageing characteristics

of porous building materials with and without surface coatings are investigated. Future aspects of this study will also include the development of ageing procedures and accelerated test methods to be applied on smaller samples. Comparison of the results from the natural and artificial ageing tests will give information on how to design short-term test methods for evaluating renderings containing water repellants.

6 Acknowledgements

We would like to thank Yxhult AB, Optiroc AB, Wacker-Chemie and Snöland Fasad AB for supplying the test materials.

7 References

Kus, H. and Kalmar G. (1998) Studies of hydrophobed plastered autoclaved aerated concrete. *Materials and Technologies for Sustainable Construction, CIB World Building Congress*, Gävle, 7-12 June, pp.285-93.

McGettigan, E. (1992) Silicone-based weatherproofing materials. *Concrete International*, June, pp.52-6.

Norberg, P. (1993) Evaluation of a new surface moisture monitoring system. *6th Int.Conf. on Durability of Building Materials and Components*, Omiya, 26-29 October, pp. 637-46.

Nygren, K. (1998) Long time performance of plastered external walls made from autoclaved aerated concrete. *Materials and Technologies for Sustainable Construction, CIB World Building Congress*, Gävle, 7-12 June, pp.387-97.

92 REACTING TO DURABILITY PROBLEMS WITH VANCOUVER BUILDINGS

M.D. Lawton

Morrison Hershfield Limited, Ottawa, Ontario, Canada

Abstract

This paper outlines how one representative of the building envelope engineering community reacted to water entry and rot problems experienced in the Vancouver Region over the latter part of the 1990's. It reviews how the nature and extent of problems were diagnosed, what remedial approaches were adopted based on available information, and identifies questions for which additional research should be directed.

Keywords: Biological decay, wood rot, mold, water penetration, remedial action

1 Introduction

Over the latter half of the 1990's it has become evident that there is a major durability problem with many of the recently constructed wood framed buildings in the Vancouver area. In the same period, Morrison Hershfield, an engineering firm specializing in building envelope durability issues, made a concerted entry into the Vancouver market. With the coincidence of these two events, Morrison Hershfield, and other building science professionals, were called on to resolve major, and immediate, building durability problems, without necessarily being comfortable that we knew everything we needed to know about the cause of the problem or that we could guarantee a solution. The situation is an interesting example of the interplay between building science and building practice in the real world.

The intent of this paper is to document how one firm reacted to the challenges and hopefully provide some insight on how building science researchers can help resolve and avoid such problems in the future.

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2 The situation

Throughout the 1980's and 90's, Vancouver went through a sustained construction boom. Good economic conditions and a very attractive location led to a rapid expansion of population with the resultant demand for housing. Housing and land prices skyrocketed and developers scrambled to meet the demand for new housing. One of the responses to demand and price pressures was the development of what could be considered a new class of buildings, three or four storey wood framed, multi unit condominiums. These buildings adapted the wood, platform frame construction method commonly used in low rise Canadian residential construction to the larger buildings. Most were clad a face sealed stucco system with no provision for drainage to the outside. The basic construction outside the insulated wood frame consists of a wood based sheathing (plywood or oriented strand board [OSB]); one or two layers of sheathing paper or polyolefin membrane and a two or three coat, 19 mm stucco with embedded welded wire mesh reinforcing.

Windows were typically aluminum framed with an integral face nailing flange for mounting. Each dwelling unit typically had one or more balconies and many were accessed via exterior walkways.

As the 1990's unfolded it became evident that there were problems with some of these wood framed condos. After very few years of operation, the walls in some of the buildings were found to be suffering from major degradation and rot of the structural wood frame. It soon became evident that the problems were wide spread and could not simply be blamed on poor construction of few buildings. If fact, there are numerous cases where major repairs were undertaken to eliminate apparent construction deficiencies only to have the same areas suffer from similar problems in two or three years.

Unit owners and their condominium corporations were desperate. They were watching their investments, and dreams, rot away. These owners and their lawyers turned to the engineering community asking what was the problem, how to fix it, and who was to blame. Eventually, as the full magnitude of the problem unfolded, the media and politicians got involved. There were editorials written and government commissions struck asking who was at fault and who was going to fix it. Eventually building research agencies focused on the specific problems looking for causes and potential solutions.

We are now well into this process, but five years ago there was much we didn't know. How extensive were problems with individual buildings? How wide spread were the problems? What mechanisms lead to the moisture collection? Why did the problem center in the Vancouver region (was it a function of the climate, building design, materials used or construction methods and quality)? And, most important from our perspective, how could the damage could be repaired and the chances of reoccurrence eliminated?

3 Diagnosing the problem(s)

As with many of our colleagues and competitors, Morrison Hershfield was asked by clients to investigate the extent and cause of problems in individual buildings. This was done primarily through the low-tech methods of visual review, moisture probing with resistance type moisture meters and creating test openings.

The ultimate goal was tó map the areas of high moisture content and wood rot. A very important step that was needed was to compile information on individual buildings to assess the significance of the many issues that could be playing a role. This was done in a 1996 project carried out for Canada Mortgage and Housing Corporation that resulted in a publication *Survey of Building Envelope Failures in the Coastal Climate of British Columbia*, (MHL 1996). This project which focused on problems with walls, was a collaborative effort of a number of consulting firms and agencies active in the Vancouver area. Files on forty-six buildings (thirty-seven with a problem history and nine without) were reviewed using a consistent data collection format that addressed construction materials and systems used, and the symptoms, extent, location, and perceived cause of the problems. The study was intended to find common factors to failures being experienced, and was not a statistical review of the extent of problems within the building stock.
Some of the key findings of the survey were:

- All the problems noted in these buildings were attributed to exterior water penetration and none were attributed to condensation of interior moisture.
- Exposure was a major factor, both in terms of orientation with respect to weather and the presence or lack of protection provided by overhangs above walls.
- While it is necessary to be cautious about drawing statistical information from the study because of the sample selection method. there appeared to be a disproportionately high number of problems with face sealed stucco systems. However, similar problems were found with other cladding systems including wood and vinyl siding.
- 90% of the problems were associated with interface details (windows, balcony and exterior walkway connections, penetration through the walls for wiring, vents etc. or where elements such as handrails penetrated the cladding to connect to the structure).
- Very few of the problems were associated with the clear walls without penetrations or openings.
- The material used for exterior sheathing did not appear to be a major factor. OSB sheathings, which were more commonly used in the total sample, were used on a disproportionately higher number of problem walls with but plywood sheathing was also used in walls that exhibited major problems.
- While the sample size of walls using polyolefin membrane type sheathing paper was low, we found no indication that these membranes were better or worse than more commonly used asphalt impregnated sheathing paper.

We would note, retrospectively, that in a number of our recent projects, we have identified major rot occurring in sections of stucco clad walls that do not have penetrations and interface details. While water penetration at the interface details is clearly the predominant class of water entry paths, there is likely a systematic problem with the face sealed stucco wall cladding systems as constructed. However, most of the problems were straight forward cases of water penetration. These problems are not new or unique. However, in Vancouver, the consequences were magnified by a combination of environmental factors and the incorporation of wall and window systems that historically had been well protected by eaves and overhangs in an architectural style that eliminated that protection and added many high risk elements such as balconies walkways and wall penetrations.

Even problems with windows were not new. In 1984 Max Baker and Ted Blackall discussed problems with Vancouver windows, (Blackall and Baker 1984). Virtually identical windows still dominate the market - and they still leak.

We would also note that this work addressed structural deterioration of material - not health issues related to the presence of toxigenic molds.

4 Development of remedial approaches

While the foregoing work was not as comprehensive as it might be, it provided guidance in the development of remedial design solutions for buildings in which repair could not be delayed due to the extent of structural degradation. The nature of the problems required that exterior cladding over degraded walls had to be removed and degraded structural wood element elements replaced. A key question was what cladding should be used to replace the original. It was clear that any lasting remedial design for the walls in question had to provide much greater protection from water penetration, particularly at interface details and, if possible, provide some measure of "forgiveness" by allowing removal of any water which did enter the wall.

In dealing with remedial projects, we were typically constrained from calling for major changes to architectural design, a factor that had a major impact on the extent of the problem.

Some important remedial design features were effectively forced on designers in the City of Vancouver when the City introduced a 1996 building bylaw (96-02) that required any future stucco systems be constructed with "*a vertically- strapped drainage cavity*" (the building code requires that "strapping" be 19 mm) or some other "*self furring, non wicking drainage medium*" or some "*other acceptable means of creating a cavity*". The same bylaw required that the cavities be drained to the outside at each floor level and provided specific requirements related to the design, detailing, and installation of a variety of elements such as flashings. This bylaw was only directly relevant in Vancouver proper, but its impact on design was felt over the entire region.

This bylaw effectively mandated systems that used the rain screen principle, (Chown 1997), (We concur with this approach). The cavity behind the stucco created a capillary break between the sheathing and structural elements and provided a drainage cavity which, if properly detailed, provided for removal of any water that did penetrate past the stucco.

There were still many details that needed to be worked out. A great deal of effort went into the development, evolution, drawing, specification and ultimately field review of construction details intended to limit the amount of water running down the face of the building, limit water penetration through the rainscreen and the interior moisture barrier at interface details, and provide for drainage of water that did penetrate past the rainscreen.

The complexity and installation cost of some of these details go well beyond what is common practice in other regions of Canada. For example, the prevalence of problems at window openings has forced us to adopt the philosophy that all windows used in the Vancouver market do, or will, leak through or around the frames into the wall system below. We therefore made it a standard retrofit practice to remove all windows and install sub-sill flashings, typically made up of self adhesive membranes, which intercept this water and drain it to the outside directly or via the drainage cavity.

Our remedial designs also incorporated design features which we were not sure actually provided significant benefit. For example, our standard details provide venting at the top and bottom of each storey high wall cavity. We are well aware of the debate over whether this top and bottom venting approach improves moisture removal.

Unfortunately, by not knowing what was "good enough" or what provided the most significant benefit, we were forced to employ everything we could think of that may help. Our clients could ill afford to waste resources on unnecessary complexity or design features. Neither could they afford the consequence of not doing enough.
Specific details that we employ have evolved over the years based on observation, application of building science principals and specific project needs. As an example, our method of treating window openings has evolved. The first reaction to the prevalence of problems at window openings was to wrap the sill, jamb and head of the window openings with impervious membranes. We now question this approach based on observations from older buildings in other maritime climates. We have noted that where impervious membranes were applied directly to the outside of wood elements, there appeared to be an accelerated rate of biological decay of the wood at the interface of wood and membrane. This probably only occurs when there is a significant source of moisture to the insulated cavity of the wall, but it did force us to recognize one more trade-off that had to be considered. We currently use impervious membrane where the protection against wetting appears to be much more important than the resistance to drying, for example, on horizontal surfaces such as the sill of window openings. Where orientation allows water to shed by gravity to a drainage point such as the jambs of the opening, we prefer to use a permeable sheathing paper as the internal moisture barrier.

5 Research

In Canada, there has always been a strong link between the consulting community practicing in the field and government research agencies which carry out or fund building research such as Canada Mortgage and Housing, National Research Council Canada and Natural Resources Canada. CMHC and NRCC notably have launched major research initiatives focused at resolving the type of problems being experienced in Vancouver. Some of this work is the subject of other papers at the 8dbmc. Morrison Hershfield has acted as a research contractor on some of this work and contributed its experience to others. We have also carried out research that is not necessarily duplicated by others.

5.1 Drying of stucco walls

One project worth noting is an experiment testing the ability the insulated cavity of stucco clad walls to dry. This responded to the imposition of the City of Vancouver's building by-law requiring vertically strapped cavity under stucco or "*some other acceptable means of creating a cavity*". A number of alternative methods of providing a drainage cavity in stucco systems have been proposed and applied on buildings, including narrower cavities and the use of drainage materials such as the dimpled plastic drainage boards or mesh type drainage mats. Assessing whether these alternatives were equivalent to the mandatory strapping requires an evaluation of their ability to resist water penetration into the walls and also some evaluation of the ability of walls to dry out any moisture that did penetrate into the insulated cavity. Morrison Hershfield was aware of ongoing research addressing the wetting side of the equation, but had no knowledge of the comparative ability to dry out moisture. We undertook an experiment to assess drying characteristics, (Morrison 1999). This consisted of a chamber study which measured the response of stucco clad wall panels. In this project, five wood frame panels were tested simultaneously. One was clad with a face sealed stucco system (no cavity) and the others had drained cavity systems of different designs. A known quantity of water was injected into the insulated cavity of each test panel and the moisture content of framing and sheathing monitored for a period of 5½ months while the test panels were subjected to a temperature and vapour pressure gradient. The effects of wind and solar radiation were not simulated.

The key findings of this project were:

- Drying in all panels was very slow and there was no significant difference between the test panel without a drainage cavity or those with the various designs of cavity.
- The water injected into the cavity flowed to the base of the panels and got absorbed in unpredictable, but explainable, patterns in the lower portion of studs, sheathing, and the bottom plate.
- The moisture tended to stay where it was absorbed. It did not distribute throughout the panel.

- Areas that obtained a high moisture content after water injection, maintained a high moisture content for a long time. One can surmise that a consequence of a local water entry point could be a local rot problem. Improved overall drying capability in the wall may not avoid this.

This was not a very sophisticated experiment and more detailed experimentation is clearly warranted. We would also comment that the value of modeling studies that cannot simulate the response to an intermittent and local source of water is of dubious value.

The results of our experiment should have major impact on our remedial design practice. In our opinion, the results indicate that if a significant amount of water gets to the insulated cavity, deterioration is virtually inevitable, at least with the fully sheathed walls that are the norm in the Vancouver area. The presence of the rain screen cladding system does not appear to increase drying potential and therefore the "forgiveness" of the wall system to water penetration into the insulated cavity. While we are convinced of the merit of rainscreen wall designs, we note that a very high proportion of the problems found in Vancouver were associated with elements that effectively bypass the rainscreen cavity. It remains critical that these elements be detailed to eliminate water entry into the insulated cavity of the wall.

5.2 Mold issues

Because of our past experience with mold research, (Lawton 1998), we were well aware that the same conditions that lead to wood rot would very likely lead to the growth of toxigenic molds. We established a policy of collecting samples of mold growth found in test openings. The results have not been comforting. Toxigenic species including Stachybotrys chartarum (atra), Aspergillus versicolor, and Trichoderma harzianum have been identified in a high proportion of the bulk samples we have collected since instituting the policy. Sampling of airborne viable particulates using Reuter Centrifugal Samplers (RCS) has indicated that mold growth in buildings, probably associated with the moisture troubled wall systems, is contributing to occupant exposure to toxigenic molds.

This issue has not yet made a significant impact on efforts to resolve the problems with the moisture troubled buildings in Vancouver. In Canada, we do not yet have legislation identifying specific requirements for dealing with toxigenic molds and available guidelines, (Fungal 1995; Assessment 1993 and Draft Guide 1998), were created envisioning situations where mold problems were both relatively small and inside the buildings. The procedures called up in these guidelines are modeled after those required by North American legislation for asbestos removal. The basic assumption used in developing these guidelines may not be appropriate in our situation. The area of mold growth is large, some unknown proportion is made up of toxigenic species and remedial action is generally carried out from outside the building.

As it becomes more widely recognized that toxigenic molds are present in these buildings, there will be a significant impact on future restoration work. Mold concerns will affect the need and timing of remedial action and extent of material removal. Work methods must incorporate provisions to protect workers and the occupants of the building during and after remediation. As practitioners we are in need of rational guidance on evaluating the degree of concern and the appropriate methods of addressing the real risks. This needs better risk evaluation and research on the effectiveness of alternate methods of isolating and cleaning up toxigenic molds.

6 Spreading the knowledge

While there are still many unknowns which need to be addressed by additional research, passing on the best available current knowledge is of clear benefit. CMHC has taken the lead in this. It funded the development of a Best Practice Guide for Wood Framed Envelopes in the Coastal Climate of British Columbia, (MHL and RDH 1998), and very recently has initiated a project to develop a best practice guide for restoration work. The first document, now published, was another collaborative effort with Morrison Hershfield being the prime contractor. Input and review by others was extensive. A host of different parties were involved and a great deal of discussion, debate and outright argument ensued. The end result is a document focusing on rain penetration control with many illustrations of specific construction details. In our opinion, this document best defines what the industry currently believes is necessary to avoid the types of problems that have been the subject of this paper.

Most of the construction details provided in the Best Practice Guide are based on the application of rainscreen cladding to conventional platform frame construction. However it also includes details which differ dramatically from current building practice in Canada. As an example, details are provided for a system that employs no insulation in the structural frame of the wall, an impervious air/vapour/water barrier mounted on the exterior of the sheathing, insulation outside the membrane and a drainage cavity between the insulation and cladding. We are aware of several projects where this approach is being applied in new and remedial projects.

7 Where are we now

This paper outlines a reaction to a crisis. In a forum such as the 8dbmc it is worth taking a pause and identifying some of the questions we the practitioners still have. This should be a basis for future research. Some of our most fundamental questions include

- Do we really have a method of lasting repair? Are the measures we have developed and implemented going to provide a lasting solution to a major durability problem with wood framed buildings.

- Are we over designing and wasting the resources of people who can, frankly, ill afford to have that happen?
- Why can't we get windows that truly work to keep the water out of the wall? Considering the cost measures we use to prepare window openings, it would seem that we could afford a better window.
- How do we react to the presence of toxigenic molds? We cannot ignore the health concerns but, on the other hand, our clients can ill afford to overreact. We need clear guidelines to identify the extent of removals, the nature and degree of worker protection required, what measures are required to protect occupants during construction and after, and what are the most effective methods of clean-up.
- What's next? The problems with buildings in Vancouver were not limited to face sealed, stucco construction on wood framed buildings. The Vancouver building stock contains a number of face sealed building systems such as EIFS, window wall and metal panel systems, and other moisture sensitive wall systems including metal framing with gypsum sheathing. The three and four storey wood framed condominiums that we have been focusing on may simply be the canary for a much broader problem to come.

As should be evident from this paper, we are firmly convinced, based on general principles, that claddings systems which make use of the rainscreen approach are superior to systems which do not incorporate a drained cavity. In Vancouver's wet climate, rainscreen cladding systems, combined with great attention to ensuring that drainage and waterproofing details eliminate water penetration to the structural elements, are the only solution we can currently recommend for walls that are susceptible to water induced decay, or growth of toxigenic molds. This is true for both new or remedial construction. The one possible exception is where the wall is very well protected from wetting by large overhangs.

We are concerned that our current designs are overly complex and recognize that there may be ways of providing the required performance with more cost effective systems. Even with drained cavity, rainscreen systems there are many possible variations in design. We do not currently have good methods of evaluating the performance of various systems or defining acceptable performance criteria.

Our experiences in Vancouver have shown that, as an industry, we have limited ability to deliver or even understand the function of wall systems that meet one of the most fundamental envelope performance requirements, keeping the rain out. To accept alternate methods of rain penetration control we would have to be provided with far better information or tools to assess long term performance in real environments.

There are a host of specific research questions we would like answers to. Some we consider high priority include:

- What are the safe limits to the amount of water that can penetrate to the interior moisture barrier?

- What are the safe limits of the amount of water that can penetrate into the insulated cavity(zero?)?
- How compact can you make a drainage cavity and have it provide an adequate capillary break and drainage path?
- Does venting drainage cavities top and bottom significantly increase drying of the insulated wall?

There are some others which may not have been definitively answered but our experience in Vancouver indicates that the answer is not likely to be significant to building practice. These include:

- What material characteristics for stucco are desirable and how do we get them?
- What are the ideal characteristics with respect to water resistance and vapour permeability of the materials to be used as the interior moisture barrier (drainage plane)?
- Do solar and wind effects make a significant difference in drying rate?
- What would be the implications of using an air permeable exterior sheathing?
- Is drying to inside a rational concept?

Answering some of these questions and resolving some of the identified concerns is going to need careful application of good building science and research in future construction and when correcting problems with the past.

8 References

Morrison Hershfield Limited (1996) Survey of Building Envelope Failures in the Coastal Climate of British Columbia, Research Report available from Canada Mortgage and Housing Corporation, Ottawa

Blackall T.N., Baker M.C. (1984) Rain Leakage of Residential Windows in the Lower Mainland of British Columbia. National Research Council Canada, BPN 42

Chown G.A., Brown W.C., Poirier G.E. (1997) Evolution of Wall Design for Controlling Rain Water, National Research Council of Canada, CTU No.9

Morrison Hershfield, (1999) Stucco Clad Wall Drying Experiment, available from Canada Mortgage ad Housing Corporation, Ottawa

Lawton, M.D., Dales. R.E., White J. (1998) The Influence of House Characteristics in a Canadian Community on Microbiological Contamination, Indoor Air pp 2-11

Lawton M.D. (1998), Moldy Houses - Building Science Lessons from the Wallaceburg Project, Proceedings of the Thermal Envelopes VII Conference, ASHRAE

Fungal Contamination in Public Buildings – A Guide to Recognize and Management (1995), Health Canada

Assessment and Remediation of Stachybotrys Atra in Indoor Environments (1993)

Draft Guide from *Toxic Mold Cleanup Procedures (1998)* CMHC

Morrison Hershfield and RDH Building Engineering (1998) Best Practice Guide for Wood Framed Envelopes in the Coastal Climate of British Columbia,

93 DYNAMIC BUFFER ZONE — DBZ

C.B. Makepeace,[1] R. Ogle,[2] and R. Nelson[3]

[1]Alberta Public Works, Supply and Services, [2] Building Science Engineering Ltd and [3]Ray Nelson & Associates, Edmonton, AB, Canada

Abstract

The *Dynamic Buffer Zone* (DBZ) is not a new concept for retrofitting problem buildings. The implementation of this approach was performed on an existing high humidity facility, with an exterior glass and aluminium curtainwall cladding, to eliminate the problem of icicles which formed on the exterior cladding. The icicles created a safety concern when they fell from the facade. Another case study will also be presented for the retrofit of the DBZ in a historic masonry building to eliminate condensation problems.

Keywords: Building envelope, condensation, air barrier, rain screen, dynamic buffer zone, mechanical pressurization.

1 Introduction

There has been much discussion since the 1950's regarding how to provide a separation between interior and exterior environments in the cold Canadian climate. Part 5 of the 1985 National Building Code, "Wind, Water and Vapour Protection", has outlined the minimum requirements for this separation. Section 5.3.1. discusses air barriers and their location in the building envelope. Reducing air leakage by incorporating a continuous air barrier in the building envelope design is fundamental in the successful performance and durability of the building envelope. There are a variety of approaches which have been used to achieve airtightness in the building

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envelope, with some being more successful at fulfilling this requirement than others. Part 5 does not restrict approaches to airtightness, but states that "the assembly shall be designed to provide an effective barrier to air exfiltration and infiltration, at a location that will prevent condensation within the assembly......".

Condensation and moisture build-up within the building envelope may cause the materials which make up the construction to deteriorate prematurely. Such moisture can also promote the growth of molds which can pose a health risk. Dripping water, icicles and staining appearing on or from claddings may be the first visible signs of moisture problems within the wall fabric. Wetting of the cladding and then freezing temperatures may result in spawling problems of claddings .

The physics of condensation within the building envelope is quite complex and quantification of the phenomenon using mathematical computer models is the subject of some research today. The basic factors which affect the process have been known for some time and include but are not restricted to the following: indoor and outdoor temperatures, material properties such as air and vapour permeability, building envelope construction details, indoor relative humidity and indoor/outdoor air pressure differences. Condensation on or within the building envelope occurs when water vapour is transported to a location that is at or below the dewpoint temperature.

Early researchers believed that the primary driving force in this transport was diffusion, a process dependent on vapour concentration differences and materials' resistance to vapour flow. This idea spawned the use of a 'vapour barrier' in the building envelope. Researchers now realize that while vapour diffusion can be a factor in some situations, the major reason for the amount of condensation in the construction of walls during the colder seasons is the outbound mass flow of moisture-laden air through the building envelope due to air leakage.

There are three separate air pressure loads across the building envelope which cause air leakage. Wind, stack or chimney effect and mechanical pressurization have been documented in papers and the Canadian Building Digests produced by the National Research Council of Canada (NRC). William Rose provided an excellent summary of the history of condensation problems and ventilation requirements in attics in a paper presented to Thermal Envelope VI Conference (Rose 1995).

Problems still may occur in some buildings having high relative humidity where even a small amount of air could result in moisture being transported into the building envelope construction. Older buildings where the interior environment is changed to provide humidity may also experience problems they never had. The Dynamic Buffer Zone (DBZ) concept can extend the durability of the building envelope for these types of situations.

First we should look at some of the history behind the problem of condensation and how the use of mechanical pressurization was viewed by the researchers. To do that I will provide excerpts from William Rose's paper (Rose 1995).

The first explicit statement that air leakage could be a major factor in building envelope condensation problems may have been by Neil Hutcheon of NRC, in 1950, in an unpublished report concerning a condensation problem at a Saskatchewan hospital. Hutcheon noted frost accumulation 10 times greater than predicted by normal vapour transmission and concluded, *"It seems necessary to assume some other mechanism for vapour migration than the usual one of vapour diffusion under vapour pressure differences, to explain the rate of moisture transmission. It is possible that leakage of warm, moist inside air outward and upward, carrying water vapour can account for this."* (Rose 1995). Hutcheon also discussed air leakage as an important mechanism for moisture transport, as distinct from vapour diffusion, in a paper for ASHAE in 1958 (Hutcheon 1958). With regards to pressure differences induced by building mechanical systems, Grant Wilson of NRC stated in 1961, *"Buildings are sometimes pressurized by a substantial excess of supply over exhaust air. The purpose of such pressurization is to reduce infiltration, presumably to overcome drafts and prevent the entry of dust." "(Mechanical) pressurization magnifies condensation problems that result from exfiltration of air and the practice is of doubtful merit in most Canadian climates. Instead, more attention should be given to increasing the air-tightness of the warm side of buildings. In general, humidified buildings should not be pressurized, and provision of a small suction in such buildings might be advantageous if condensation problems are anticipated."* (Wilson 1961). Thus even early on it was evident to researchers that mechanical systems might be used to reduce or eliminate condensation problems due to air leakage.

In 1965 in a paper entitled *'Moisture Accumulations in Roof Spaces Under Extreme Winter Conditions'*, presented to the RILEM/CI Symposium on Moisture Problems in Buildings, Dickens and Hutcheon identify air leakage as a mechanism of moisture accumulation. They go on to describe some of the mechanical ventilation of roof spaces to avoid condensation problems. *"The building had experienced severe condensation problems prior to the installation of fans which were used initially to dry out the wet roof structure. The fans were also kept running the following winter to counteract air leakage into the roof space from the heated area by slightly pressurizing the space.... Ideally the pressure should be sufficient to prevent air flow into the roof space from below without causing significant reverse flow of cold air into the building."* (Dickens and Hutcheon 1965).

In another paper presented at the same 1965 symposium, Trygve Isaksen from the Norwegian Building Research Institute identified air leakage as a cause of condensation within roof spaces and offered three possible solutions to the problem in a flat wooden roof. He states: *"A cheaper solution is to use a small medium pressure fan to blow in cold air above the insulation in the existing roof, creating a super pressure in the air space to prevent air leakage from beneath."* (Isaksen 1965).

Both papers pointed out that one method of preventing humid interior air from leaking into an air space within the building envelope was to pressurize the space with outside air by mechanical means.

Kerby Garden wrote in 1966, in a Canadian Building Digest concerning swimming pools: *"The best enclosure design may be one in which the exterior enclosure elements are not exposed to the severe environment of the swimming pool* *space. This can be accomplished by incorporating a heated space between the interior surfaces and the main enclosing elements and ventilating it with low humidity air. In this case the inner layers of materials will be required to resist the passage of air and vapour, but will not be subjected to a temperature gradient. Any moisture that might leak into the space will not raise the humidity appreciably because it will be removed by the ventilation process. This space can be as little as is required for proper air movement or it can be wide enough to perform as a corridor."* (Garden 1966).

Here the concept of a buffer zone between the humid interior and the exterior envelope is introduced. The buffer zone is ventilated with low humidity warm air which flushes away any moisture that might leak into it. The idea of pressurizing this space to prevent air leakage was not discussed. The use of a buffer zone to prevent condensation in high humidity buildings had actually been used in practice before this time. For example, Ernest Cormier an architect working in eastern Canada in the 1950's and 60's, had used heated air between single lites of glass in a Government Printing Office to prevent condensation (Handegord 1998).

When the idea of a conditioned buffer zone was combined with the idea of mechanical pressurization to prevent air leakage the result is what is now termed the Dynamic Buffer Zone (DBZ).

The first description of the DBZ may be in the book *'Building Science for a Cold Climate'* (Hutcheon and Handegord 1983). *"When the insulation is located between two impermeable layers it becomes possible, in principle at least, to control air and vapour flow in walls and roofs by pressurizing this intermediate space mechanically using dry air. The impermeable layers may have to have a high degree of airtightness to avoid loss of energy by leakage. This approach is attractive in extreme situations such as high-humidity textile mills and swimming pool buildings where heated, ventilated air under pressure is needed and can be supplied by way of the intermediate space."* The fact that the authors say, "at least in principle," suggests that few built examples were available at that time.

2 Dynamic Buffer Zone -- (DBZ)

While the code requires air tightness in building envelope assemblies subjected to a temperature differential, a differential in water vapour pressure and a differential in air pressure, the high degree of airtightness required to eliminate or at least control condensation problems in some humidified buildings may not be possible. This is true in existing building retrofits, the repair of building envelope failures, and even in some cases of new construction.

The Dynamic Buffer Zone concept introduces, between the inner and outer surfaces of the building envelope, heated cold outside air containing little moisture. The DBZ concept attempts to eliminate air leakage into the building envelope by pressurizing this area. It is not intended to ventilate this space to dry it out. Heating of the outside air is only necessary to a degree where inner surfaces will not become excessively cold, and where infiltration through imperfections in the inner surface will not result in condensation. Such infiltration of cold air could result in condensation on cooled surfaces, freezing of pipes and comfort problems. The DBZ air temperature is much closer to that of the interior. This can in turn reduce stack effect in the cavity and reduce air infiltration through the wall at lower elevations.

Obviously a certain degree of air tightness is necessary in both the inner and outer surfaces of the wall to maintain positive pressure within the air space between them without presenting other problems of noise and excessive cost.

In tests on buildings in Ottawa, it was found that the exterior mass masonry and interior finish lath and plaster were sufficiently air tight to achieve the pressures required (Quirouette 1977). By contrast the retrofit of a hospital aluminium and glass curtainwall to be described further in this paper required extensive sealing of the exterior and some interior sealing to achieve the required air tightness.

The fans which operate the DBZ should be fully automated to see economies in the design and to control the system performance during changes in weather conditions. One control strategy is to initiate the operation of the system based on outside air temperature while another is to activate the system when the moisture content of the air in the buffer zone increases to above outside levels. The writer prefers the temperature approach because temperature is easier to monitor.

In 1986, based on discussions with Kerby Garden, Alberta Public Works eliminated severe condensation problems within the terra cotta dome of the Alberta Legislature Building by pressurizing the air space below the terra cotta with heated dry exterior air. This was a direct application of the DBZ concept (Ogle et. al 1995).

In 1996 and 1997 building envelope consultants Michel Perreault and Rick Quirouette implemented the DBZ in several heritage masonry buildings that were being renovated, including new operating conditions with high interior humidities which potentially place the masonry in peril. Pressure, temperature and humidity data were obtained from test sections of the walls and control strategies for the mechanical system were devised (McGrath and Perreault 1996) and (Quirouette 1997).

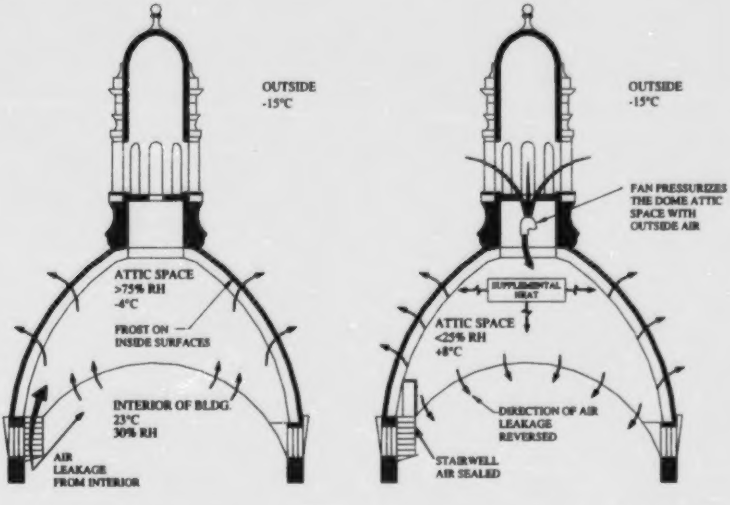
3 Case studies

3.1 Case study No. 1 Alberta Legislature Building (1986)

In the early 1980's it was noted that the terra cotta tile dome of the Alberta Legislature had, in one of the 8 segments, bulged from its original curved plane. One of the concerns addressed in the subsequent structural and building envelope studies was the severe condensation that was occurring on the inner surfaces of steel and terra cotta book tiles.

The dome is approximately 55 feet in diameter and is composed of an outer terra cotta decorative or finish tile and an inner book tile supported on a steel frame.

From the same steel frame, an inner dome is suspended, creating a large attic space. Leakage of humid interior air through a stairwell access and through the construction was resolved by limited air sealing of the known air leakage problem locations and implementation of the DBZ concept (Figure 1).


OUTSIDE
-15°C
ATTIC SPACE
>75% RH
-4°C
FROST ON
INSIDE SURFACES
INTERIOR OF BLDG.
23°C
30% RH
AIR
LEAKAGE
FROM INTERIOR
OUTSIDE
-15°C
FAN PRESSURIZES
THE DOME ATTIC
SPACE WITH
OUTSIDE AIR
SUPPLEMENTAL
HEAT
ATTIC SPACE
<25% RH
+8°C
DIRECTION OF AIR
LEAKAGE
REVERSED
STAIRWELL
AIR SEALED

ORIGINAL CONDITION
(DOME PRESSURE ?)

CAVITY PRESSURIZATION WITH OUTSIDE AIR
(DOME PRESSURE +30 Pa)

Fig. 1: Typical winter conditions in the dome before and after repairs

A 500 cfm fan, was used to extract cold exterior air from the cupola at the top of the dome. The air was ducted into the attic space and around the attic space. A glycol heat pump was used to heat the air being introduced. Two heaters with a 40 kW total output were installed in addition to augment the air temperature when necessary. The fan pressurized the attic space to about 30 Pa. during cold conditions, while the heaters maintained the space at a minimum of 5°C.

The DBZ eliminated the condensation problem without affecting the aesthetics of the stabilized terra cotta dome. Actual measurements of typical conditions within the attic during a winter day before and after implementation of the DBZ are noted in Figure 1.

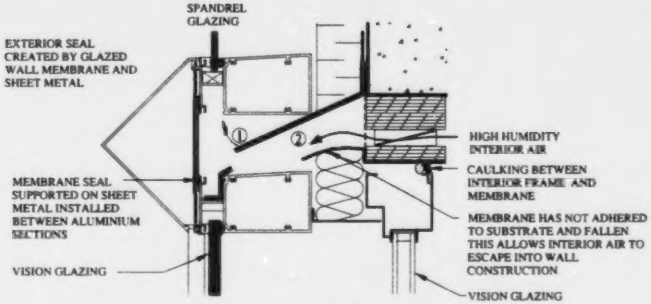
3.2 Case study No. 2: Aluminium and glass curtainwall Alberta (1997 - 98)

The interior environment of this problem building was operated at a positive pressure relative to the exterior, of about 40 Pa and a relative humidity at approximately 35% during the winter. Operational requirements dictated that these operating conditions could not be changed.

The construction of the building envelope consisted of a finished inner wall of drywall, metal furring, cast-in-place concrete (CIP) or concrete block. Openings within the CIP concrete portions of wall were created for the installation of double glazed sealed units in aluminium frames. Adhered to the exterior of the CIP concrete and concrete block was a peel and stick membrane. The intent of this design was for the membrane and backup wall to provide a continuous air barrier system. Rigid insulation was mechanically fastened tight to the membrane. The exterior curtainwall cladding consisted of a single lite of vision glass or single lite of opaque glass flush glazed, in a so-called "open rainscreen" aluminium frame. The cladding was designed to provide aesthetic feature horizontal covercaps, and a uniform reflective appearance from the exterior. It was only designed to shed and drain rainwater, not to provide any air tightness.

In the first winter of operation condensation appeared on the inner surface of the exterior vision lites of glazing and icicles formed on the exterior of the covercaps and glass. During periods of warm weather or when the sun created enough heat to reduce the adhesion of the ice on the metal and glass of the exterior, large pieces would depart the facade and ricochet off lower horizontal covercaps away from the building facade. This created a significant safety hazard to pedestrians within a zone of 9'-0" (2.74 m) from the building perimeter.

The cause of the condensation was determined during the installation of the DBZ, to be flaws in the membrane system between the cast-in-place concrete and the window frame (Figure 2). This allowed highly humidified air to escape into the cold cavity space between the membrane and the exterior glass and aluminium curtainwall where it condensed. During warming periods water would drain to the exterior through weeping provisions provide in the aluminium curtainwall framing. Under cold conditions the water would form icicles on the cold exterior surfaces.


SPANDREL GLAZING
EXTERIOR SEAL CREATED BY GLAZED WALL MEMBRANE AND SHEET METAL
①
②
HIGH HUMIDITY INTERIOR AIR
CAULKING BETWEEN INTERIOR FRAME AND MEMBRANE
MEMBRANE SEAL SUPPORTED ON SHEET METAL INSTALLED BETWEEN ALUMINIUM SECTIONS
MEMBRANE HAS NOT ADHERED TO SUBSTRATE AND FALLEN THIS ALLOWS INTERIOR AIR TO ESCAPE INTO WALL CONSTRUCTION
VISION GLAZING
VISION GLAZING

① THE THROUGHWALL FLASHING HAS BEEN CUT AT THE OUTER EDGE AND LIFTED TO EXPOSE THE AIR SEAL MEMBRANE.

THE RETROFIT IS TO RELOCATE MEMBRANE AT THE PLANE OF AIR SEAL AND THEN URETHANE FOAM ② THE VOID BETWEEN THE MEMBRANE AND THE THROUGHWALL FLASHING TO RETAIN MEMBRANE.

Fig. 2: Head detail between vertical millions of visions windows

During the winter of 1997-98 a 6000 sq. ft. full scale test section of the wall was selected to be converted to a DBZ to determine if this concept could alleviate the safety issue. Two masonry clad stairwells isolated the test area from the other areas of glass and aluminium cladding.

The test section of wall was scaffolded on the exterior to facilitate the sealing of the exterior glass and aluminium curtainwall framing and for the cladding to be air sealed back to the original air barrier system of the building. Construction took place from November to January. The scaffold was tarped and the work platforms heated. The various details necessary to provide this air tightness were complicated by the existing aluminium framing system configuration and provided the design team and the retrofit contractor with several challenging issues. Removal of the opaque spandrel glazing was necessary to modify the vertical aluminium tubes and flashings of the original system. The vertical tubes were used as air ducts. Holes cut into the tube walls of these allowed for horizontal air distribution. The gaps between aluminium sections were bridged with galvanized sheet metal to provide support for the installation of a high temperature peel and stick membrane that provided the seal. The design only required that sufficient air tightness be achieved in the exterior cladding so that an air pressure could be created in the cavity.

The existing configuration of the physical plant central air system of the allowed the design team to install a new small duct prior to the humidification section of the unit. This provided filtered preheated air for the DBZ system with a low moisture content. A VAV box and an additional heating coil were installed in the ductwork to control the flow rate in the DBZ and provide additional heating of the air if it was necessary (to date no additional heating has been required). The insulated ducting was installed from inside the penthouse, across the roof, to a new horizontal parapet duct that was designed to distribute the air at the top of the test section of wall (Figure 3).

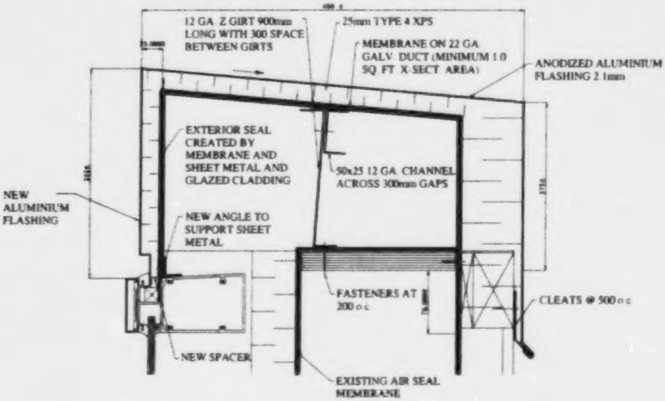

12 GA Z GIRT 900mm LONG WITH 300 SPACE BETWEEN GIRTS
25mm TYPE 4 XPS
MEMBRANE ON 22 GA GALV DUCT (MINIMUM 1.0 SQ FT X-SECT AREA)
ANODIZED ALUMINIUM FLASHING 2 1mm
EXTERIOR SEAL CREATED BY MEMBRANE AND SHEET METAL AND GLAZED CLADDING
50x25 12 GA CHANNEL ACROSS 300mm GAPS
NEW ALUMINIUM FLASHING
NEW ANGLE TO SUPPORT SHEET METAL
FASTENERS AT 200 o c
CLEATS @ 500 o c
NEW SPACER
EXISTING AIR SEAL MEMBRANE

Fig. 3: Parapet duct detail

Since the central air system produces a high static pressure (in the range of 1500 Pa to 2500 Pa), a pressure reducing damper (blast gate) was installed between the central air system and the VAV box. This damper was adjusted to reduce the supply air stream static pressure to a maximum of around 70 Pa. A safety sensor was installed near the top of the wall cavity. If a static pressure above 120 Pa occurs the controls automatically shut off flow at the VAV box and an alarm is sent to the building operator.

The new DBZ ventilation control devices were tied into the existing Direct Digital Control (DDC) building automation system to ensure accurate control. The VAV box flow rate modulates according to a DDC control loop that references to a static sensor located in the wall cavity near the bottom of the DBZ wall. The sensor measures the differential pressure of the cavity with respect to the adjacent interior building space. The box flow rate modulates to maintain a pressure at a minimum differential pressure setpoint of +20 Pa.

This was a pilot project and the first installation to the design teams knowledge, of the DBZ concept for a curtainwall cladding. To foster a successful outcome additional stand-alone monitoring stations were installed within the vertical height of the wall, plus one on the exterior. The exterior monitor senses the outdoor conditions. The three wall monitoring stations sense temperature and humidity, and measure the differential pressure between the wall cavity and the interior space. One of the stations also monitors the differential pressure between the outdoor air and interior of the building.

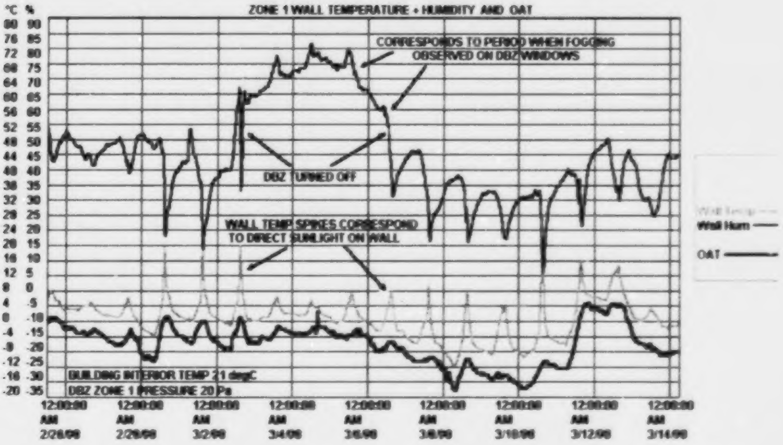


Fig. 4: Cavity monitoring of DBZ

Monitoring is performed by portable ACR brand dataloggers configured to take samples four times per hour, for a maximum of around 90 days of continuous data. Datalogger information is downloaded onto a portable laptop PC without interfering with the data acquisition process underway. Data can then be graphed and analized to review the system's performance and fine tune its operation.

Data from the monitoring system clearly shows that the approach implemented does work (Figure 4). The visible (more real) proof is that condensation is not appearing in the vision areas of the wall and no icicles are forming. Monitoring of the wall is continuing. The cost of operating the system is estimated at less than $300 CAN per year.

4 Summary

While the approach of creating air tight building envelopes has advanced, the ability to achieve high enough levels of airtightness in high humidity buildings may not always be possible, particularly in retrofit situations. Condensation within the building envelope can be detrimental to the wall components' durability. Health and safety issues are significant and cannot be ignored any longer.

To date the idea of altering interstitial wall pressures by mechanical means to control exfiltration has largely been overlooked. Some early researchers did suggest mechanically induced pressure or ventilation of air spaces within the envelope as a means of solving condensation problems. However, it is only fairly recently that the pressurization of wall cavities using dry exterior tempered air has been used. The Dynamic Buffer Zone technique has now been implemented successfully on several buildings.

The two case studies presented of existing buildings with building envelope problems have shown that durability and performance of the building envelope can be enhanced by implementation of the DBZ.

5 References

Dickens, H.B. and Hutcheon, N.B. (1965) Moisture Accumulation in Roof Spaces Under Extreme Winter Conditions. *RILEM/CIB Symposium* Helsinki.

Garden, G.K., (1966) Indoor Swimming Pool. *Canadian Building Digest 83*, Division of Building Research, National Research Council of Canada.

Handegord, G.O., (1998) Personal Communication.

Hutcheon, N.B., (1958) Vapour problems in thermal insulation. *Symposium on Thermal Insulation*, ASHAE, Minneapolis, Minn.

Hutcheon, N.B. and Handegord, G.O., (1983) Building Science for a Cold Climate. *National Research Council of Canada* (Book).

Isaken, T. (1965) Damage in wooden roofs over heated rooms. *RILEM/CIB Symposium* Helsinki.

McGrath, P. and Perreault, M. (1966) Dynamic Buffer Zone. ABN 11.

Ogle, R., Dennis, B. and O'Connor, J. (1995) Investigation and repair of a terra cotta dome in a cold climate. *Thermal Performance of the Exterior Envelopes of Buildings VI* Clearwater, Florida.

Quirouette, R. (1997) The Dynamic Buffer Zone. *The Construction Specifier*. August.

Rose, W.B. (1995) The History of Attic Ventilation Regulation and Research. *Thermal Performance of the Exterior Envelopes of Buildings VI*, Clearwater, Florida.

Wilson, A.G. (1961) Air Leakage in Buildings. *Canadian Building Digest 23*, Division of Building Research, National Research Council of Canada.

Part 5 Wind, Water and Vapour Protection (1985) National Building Code of Canada

94 FOUNDATION FAILURES IN NEW RESIDENTIAL CONSTRUCTION

R.R. Marshall

Halsall Associates Limited, Toronto, ON, Canada

Abstract

The estimated annual cost of claims to warranty programs in Canada from foundation failures is $2,134,000/year. These claims costs do not include owner or builder repairs, or the legal, and administrative costs incurred. Similar soil conditions exist in other countries, so it is likely other geographical locations are experiencing a high cost of failures. These recurring costly failures can be prevented using knowledge we have today. From the survey of 252 major claims, which were confirmed by warranty officials, the most frequent failures were due to adfreezing and swelling clays. These soil conditions caused footings, garage and stairwell components to become unserviceable.

Focusing on appropriate foundation design and proper site analysis could eliminate over 70% of the most costly failures. Building envelope designers specifying good construction practices would help ensure frequent failures are prevented. Technical guidelines have been developed for the most critical foundation elements and system solutions are available.

The bottom line is that it makes economical sense to build foundations rights the 1st time such that major structural and moisture problems can be minimized. These measures need to be implemented by the construction industry. The public should also be protected by better Building Codes and extended warranty coverages.

Keywords: causes, cost, failures, foundation, new construction, residential

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1 Introduction

This survey was undertaken on behalf of the Steering Committee for the Development of Performance Guidelines for Basement Envelope Systems and Materials of the National Research Council.

The purpose of the project is to characterize basement envelope failures using statistics provided by New Home Warranty officials in all provinces in Canada. This work adapted the approach for recording construction defects previously developed for the Ontario New Home Warranty Program (ONHWP). The ONHWP data has been extrapolated and presented in this report.

A critical aspect of the work was to attempt to get all the Warranty Program administrators participating in the project so that a national database could be created. A letter of introduction was sent out to all warranty programs. Accordingly, participation by the New Home Warranty Program of British Columbia & Yukon, The Alberta New Home Warranty Program, New Home Warranty Program of Saskatchewan, New Home Warranty Program of Manitoba, Association Provinciale Des Constructeurs D'Habitations du Quebec, and Atlantic New Home Warranty Corporation was achieved.

From the data collected the failures were characterized as follows:

- System type
- Soil type
- Envelope element
- Occurrence
- Severity
- Cause
- Remedial action
- Costs

Patterns were established for the causes for recurring defects in basements. In addition "hot spots" for failures induced by difficult soil or ground water were identified in sufficient detail to allow for specific design and construction recommendations to be made that will reduce problems. The incidence of leakage problems has also been indicated. Builders would adopt good building practice to reduce risk and expensive repairs. Technical guidelines will help the industry utilize the best solutions currently known.

According to Marshall and Martin (1993), foundation failures are the most costly defects in houses in Ontario. The cost in 1992 was estimated at $1.1 million dollars excluding administration costs and costs paid by builders. Finish, plumbing, windows and structural defects were the 2nd to 5th most costly, respectively. Comprehensive research had not been done for other provinces.

2 Methodology and data collection

When one produces a major survey, to characterize the causes of basement failures in new residential construction, the challenge is to assemble it into meaningful and actionable results. The analysis of the survey data helped to highlight 11 key areas of focus. These foundation fundamentals are presented in the conclusions of the report and if implemented will achieve improved quality, and deficient free foundation envelopes.

This survey is the essential first step, which has documented the problems in terms of the most costly and most frequent failures. By prioritizing the failures, the most suitable construction practices can be used to achieve success.

The survey methodology consisted of the following tasks:

- Identification of the number of houses enrolled in each province.
- Establish the frequency and cost of foundation failures.
- Development of a checklist of specific data including construction type, soil type, envelope elements, and cost data.
- A checklist of the characterization of basement envelope failures including detailed technical data on failure mechanism, symptom, foundation component, contributing loads, probable cause and remedial action was developed.
- The survey was conducted with the information gathered from the failure files for 1994 and 1995 from nine Provincial Warranty Programs.

The claims files were confidential and getting access to the data was difficult. It was not possible to send files that contained privileged information across the country for analysis. Due to time limitations, a detailed survey and site visits were precluded. Simplified checklists were developed to expedite the process. In the final analysis, it was necessary to group failure characteristics for similar foundation failure types.

3 Results of survey - Number and cost of failures

Based on available data, 252 structural and environmental failures had occurred in the 9 provinces surveyed. The following is a breakdown of failures reported by province for the 1994 and 1995 period.

Table 1: Failure numbers and costs

Province	**1994**		**1995**	
	Number	Cost	Number	Cost
Newfoundland	15	$44,000	5	$18,000
Nova Scotia	42	$138,360	6	$35,052
Prince Edward Island	2	$19,000	2	$13,000
New Brunswick	27	$221,000	10	$50,000
Quebec	2	$12,476	5	$45,131
Manitoba	6	$29,730	4	$18,500
Saskatchewan	5	$12,000	4	$18,000
Alberta	22	$80,000	10	$30,000
British Columbia	43	$1,222,720	42	$1,194,284
Total	**164**	**$1,779,286**	**88**	**$1,421,967**

The average for the 1994 and 1995 period is 126 failures per year. The failures must be normalized to reflect the annual number of homes enrolled in each province. However, based on the ONHWP geotechnical study, Marshall and Morrison (1995), the total number of failures on average for the 1992 to 1994 period in Ontario was 46 failures/year. This is slightly higher then the second most frequent failures in British Columbia with 42.5 failures/year.

The average cost of structural failures for the 1994 and 1995 period was $1,601,000. Based on failure data from all warranty programs including an estimated annual cost of failures in Ontario of $533,000, the annual cost of foundation failures is about $2,134,000/year. The high cost of failures is unfortunate as we have the knowledge to prevent these failures.

4 Analysis of results

4.1 Analysis by enrolment

A detailed analysis was done of the 1994 and 1995 failures to probe the severity of failures by province and to normalize the data. The number of enrolments is an important factor, as a high failure rate in an area with relatively few enrolments will lead to serious financial problems.

Table 2: Foundation failures ratio by province - 1994 & 1995

Province	Enrolments	Failures	Failure Ratio per 1,000 Enrolments
Newfoundland	1,146	20	17
New Brunswick	2,750	37	13
Nova Scotia	4,294	48	11
PEI	460	4	9
Saskatchewan	1,574	9	6
Manitoba	2,374	10	4
British Columbia	39,549	85	2.1
Alberta	19,644	32	2
Quebec	23,400	7	.3
Total	**95,191**	**252**	**2.6**

One of the main objectives of this survey was to assess claims and to reduce both the incidence and cost of repairs to the participating warranty programs. In Newfoundland 9 of 17 of failures were frost heave related. In New Brunswick 12 of 13 of the failures were caused by adfreezing. 11 of 11 of the foundation failures in Nova Scotia were adfreezing related. Saskatchewan had 3 of 6 of the foundation failures caused by swelling clay.

It is important to understand that the warranty representatives identified adfreezing as the cause of failures and field verification was not possible. Adfreezing occurs when frost susceptible soil adjacent to foundation elements forms a strong freezing bond. Ice lens formation in the soil results in heaving and an upward thrust in the structure.

4.2 Analysis by region

Halifax had the highest number of failures due mostly to adfreezing of basement stairwells.

Table 3: Locations with more than one foundation failure in 1994 – 1995

Location	Average Number of Failure/Year
Halifax (NS)	24
Kamloops (BC)	6
St. John (NB)	3.5
Regina (Sask.)	3
Calgary (Alta.)	2.5
Moncton (NB)	2.5
Gander (Nfld.)	2
Ucluelet (BC)	2
Kelowna (BC)	1.5
Lachenaie (Quebec)	1.5
Quisparisis (NB)	1.5
Saskatoon (Sask.)	1.5

4.3 Analysis by type of foundation envelope system

The data was examined to determine foundation failure occurrences of foundation failure for different types of foundation systems. The most common type of foundation envelope was non-reinforced concrete with 179 of 215 cases. Poured concrete with reinforcing was present in 25 cases and there was 10 block foundation.

A comparison of foundation failure mechanisms and foundation envelope types revealed that for adfreezing type failures, all 10 block foundations representing 100% failure by this mechanism. By comparison, only 1 adfreeze failure occurred with the 25 reinforced concrete foundations representing 4%. According to the data provided by warranty officials, there were 81 adfreezing failures in the 179 non-reinforced concrete foundations representing 45% of the total. The installation of reinforcing seems to have some benefit for reducing the incidence of adfreezing failures.

A further comparison was made of the swelling soil failure mechanism and foundation types. There were 65 swelling soil failures in the 179 non-reinforced concrete foundation envelopes representing 36%. In the 24 reinforced poured concrete foundation envelopes there were 11 swelling soil failures representing 46%.

It is evident that the installation of reinforcing, which consisted mostly of horizontal bars, does little to prevent the foundation envelope failures caused by swelling soils. Swelling pressures can be about as high as 10,000 lbs./sq. ft., which is significantly higher than the 500 lbs./sq. ft. of bearing pressure for typical residential construction. A design approach is required to prevent this type of failure such as preloading the site to consolidate weaker soils in advance of construction.

4.4 Analysis by failure mechanism

Adfreezing cases are the most frequent failure. Adfreezing problems were also the most frequent problem at 48%, according to Marshall and Morrison, (1995). Most cases relate to walkout basement stairwells and unheated garages.

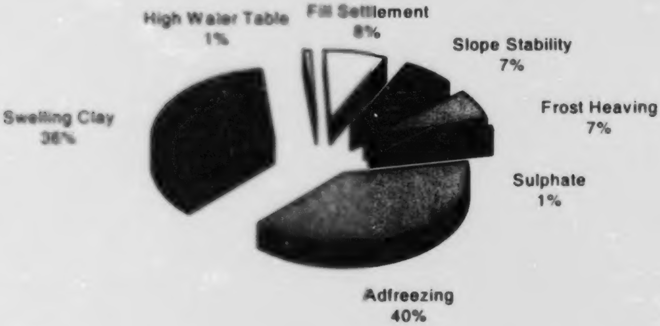

Fig. 1: Overview of failure mechanisms

The two most critical foundation failure mechanisms that must be addressed are adfreezing and swelling clay.

4.5 Analysis by severity

From the data collected it is evident the vast majority of foundation envelope failures result in horizontal and vertical cracks which bow walls, crack drywall, damage doors and impact adversely the operation of windows. A total of 172 of the 215 cases representing 80% fall into this category. The New Home Warranty Programs in Alberta and British Columbia report 13 localized structural collapse failures representing 6% of the failures investigated. No definitive conclusions can be reached from this data as many related symptoms were identified which in combination caused a severe failure. The primary symptom was cracking which would be expected.

4.6 Analysis of failures by components

Walls at 13% and exterior stairwells/steps at 26% and grade beam/slabs at 6 % in total account for 45% of foundation envelope failures. In these cases the failure mechanism is not related to the load bearing characteristics of the underlying soil but the swelling/adhesion characteristics of the surrounding soil which cause the failures.

4.7 Analysis of failures by probable cause

Overall from the data collected it is evident depth of footings is not the most significant probable cause of failure but is a factor often referred to by code users. From anecdotal information from Atlantic NHWP, depth of footing was a factor in 80% of the frost related foundation failures. Design details for frost protected foundations would achieve the most benefits. These should be in the form of guidelines which present the principals mechanisms that must be addressed with adfreezing and the typical practices which have been proven in the field to improve the performance of foundations.

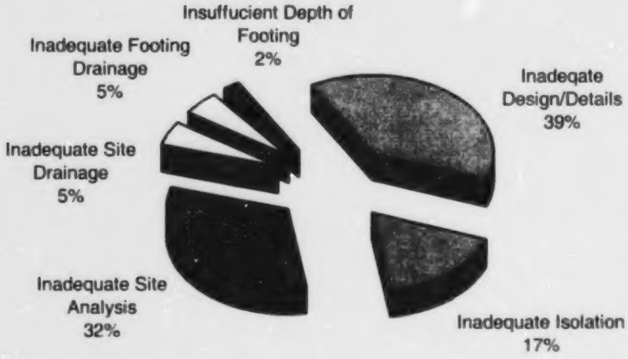

Fig. 2: Probable causes

5 Remedial actions, cost and estimated economic significance

5.1 Action and cost

The most costly remedial actions involved the installation of footing drainage, sumps, and surface drainage work. The total cost of these remedial actions was $1,866,000 for the 71 cases documented. The second most costly remedial action taken was piling under the existing foundation. The total cost of these remedial actions was $620,000 for the 21 reported cases. Interestingly, this remedial approach was undertaken in the two most western provinces with 14 cases in Alberta and 7 in British Columbia. The third most costly remedial actions involved epoxy injection combined with drainage, waterproofing, insulation, drainage layer, footing drainage and surface drainage work. The total cost of these remedial actions was $359,000 for the 72 cases documented. Insulation and footing drainage was the fourth most costly remedial action with a total cost of $92,000 for the 17 failure cases. The fifth and sixth most costly remedial actions were complete system replacement and partial replacement at $84,000 and $66,000 for the 3 and 11 failure cases documented respectively. The seventh most costly remedial action was the installation of a drain layer at $22,000 for the 4 failure cases examined.

It is evident from an evaluation of the remedial measures that in most cases one remedial action could not be undertaken to fix the failed foundation. A systematic approach was necessary in almost all cases to correct the problem. The costs per remedial case were very expensive. For example in the top 2 categories, the per failure unit cost for the footing drainage, sump system and surface drainage was $26,000/case and per failure unit cost for piling was $29,000. If the foundations were built right the first time, it is estimated the added construction cost would have been only $5,000 to $7,000 which is less than 25% of the remedial cost.

5.2 Estimated economic impact

It is essential that any measures that are implemented to prevent foundation failures be done in a cost effective context. This survey documented the cost and frequency of failures and has not examined the costs of preventing the problems. However, if investments are to be made to address these failures we need to know at least notionaly that the economic returns in the form of lower repair costs and warranty rates will be much greater than the costs of new procedures and training initiatives. A balanced and targeted approach will work best where builder information is key and the focus of education is matched to the areas with the highest risk.

A national assessment of the economic impact of various action plans was completed. Based on 100,000 new homes constructed a year, and a mandated certification inspection program for the design and field review of all foundations, the estimated cost for engineering would be $85,000,000/year or $850 per lot. (A range of fees from $200 to $1500 has been assumed). Clearly, it is not a good investment to spend $85 million to save $2 million.

What is required is a balanced preventative approach. Thus, assuming an estimated 20,000 registered builders had the failure information in this report and were provided with practical guidelines to prevent foundation envelope failures, at a cost of $10/builder, the estimated cost to new home buyers would be $200,000. This is less than 10% of the annual cost of failures. If only 50% of the failures were prevented, over $1,000,000/year in repair costs would be saved.

By combining this technical information, with a targeted educational program in the foundation failure "hot spots", builders could better manage their risks without adding to the cost of construction in all locations. Builders would also achieve an enhanced reputation and at the same time provide to their customers a better built and deficient free home.

Warranty programs should be instrumental in affecting the behavior of builders who choose to ignore the foundation failure prevention guidelines. Assuming focused hands-on workshops were held in conjunction with Home Builder local and/or Building Official chapter meetings, the cost for workshops would be minimal. The only cost other than out of pocket expenses would be for the creation of customized regional workshop modules for swelling clay and adfreezing. It would not be effective to speak on swelling soil in Halifax when adfreezing is the most important foundation failure mechanism.

6 Discussion and conclusions

- A total of 252 claims have been surveyed from 9 provinces. This provides a valid sample to characterize the causes of foundation envelope failures.
- The cost of failures of $3,201,000 or $13,000/failure is significant. The total cost could be much higher if the costs of administration, engineering and repair costs paid by builders, consumers and municipalities were also included.
- A total of 172 of the 215 cases representing 80% of foundation envelope failures result in horizontal and vertical cracks which bow walls, crack drywall, damage doors and adversely affect the operation of windows. Home owners need protection for these performance problems. Consumers especially in the Atlantic Provinces, Saskatchewan and Manitoba who have high foundation failure ratios are at the greatest risk.
- NRC through industry partners should create good practice technical guidelines, which reflect regional problems. These can be used to prevent the most common and costly problems. The guidelines and other useful resources should be referred to in the Code Appendix, Housing Illustrated Guides and the Part 9 Commentary.
- The estimated economic impact indicates it would be beneficial to invest in information dissemination and educational programs to assist builders in preventing foundation failures. This would help reduce warranty repair costs and provide Canadians with more deficient free foundations.
- Foundation failure rates in Newfoundland, New Brunswick, Nova Scotia, Prince Edward Island, Saskatchewan and Manitoba are above average in terms of claim to enrolment ratios for the 1994 and 1995 years. The leading failure mechanisms are adfreezing and swelling clay. The high cost of swelling clay, hillside slippage and collapsible soil foundation failures in British Columbia must be covered.
- It would be economical to do site analysis and incorporate better design practices for foundations built in the regional areas of Halifax, Kamloops, St. John, Regina, Calgary, Moncton, Gander, Ucluelet, Kelowna, Lachenaie, Quisparisis and Saskatoon. It would be prudent for builder/developers to ensure soils information is obtained prior to the construction of the foundations. By engaging a competent geotechnical consultant, the risk to the builder of foundation failures is drastically reduced. At the inspection stage, municipal and private inspectors should be focusing their efforts on checking for poor soils.
- It is essential improved practice methods be further refined to prevent adfreezing and better practices be identified for swelling soil foundation envelope failures. These practices must be widely distributed in all areas where these failures are known to occur directly to all builders and through local jurisdictions.
- No other reports are available to indicate whether the frequency of foundation failures is on the rise or is decreasing. Once the solutions are implemented, it is essential to measure the results. An analysis of claims should be conducted within three years to assess the impact of the measures implemented.
- The collection methodology was successful as we learned vital information from each occurrence of envelope foundation failure. This is essential to prevent future recurring defects. The checklist approach is excellent but should be shortened to two pages to help simplify their completion by technical personal. From a risk

management perspective, only failures with higher cost and high rates of frequency should be analyzed in detail. Consideration should be given to site verification of a sample of the failures.

- To ensure the most practical solutions are developed in the technical guideline, an important next step in the process is to assemble key people in a facilitated expert's forum. The face to face exchange of information would be a learning experience for all participants. The practical solutions that are identified for the most common and costly foundation failures must be incorporated into the final performance guideline.

7 Recommendations

The results of this survey clearly indicate that there are significant opportunities to improve the performance of foundation envelope systems. The recommendations, which are presented below, focus on improving the process of foundation construction.

- The most serious foundation failures from metastable soils and swelling clays are occurring in British Columbia and western provinces. The high foundation failure ratios for the Atlantic Provinces and Saskatchewan are somewhat alarming. Prevention methods for swelling clays, adfreezing, settlement, slope stability and frost heaving foundation envelope failures must be established. The leadership role of the National Research Council of Canada should be maintained to identify design principals and best available alternatives.
- Details are the key if foundation failures are to be prevented. Performance Guidelines created by NRC will be used by designers to address foundation failures. The practicality of construction must also be reflected if the details are to be adapted for use on project drawings and implemented in the field. Therefore, it is recommended that the Performance Guidelines be developed in partnership with leading builders that have experience in building in problematic areas. The guidelines must have a high degree of illustrated content to ensure users understand good engineering practice.
- Site analysis is essential. A thorough and complete geotechnical investigation must be conducted and the designer recommendations followed. The analysis will allow for the specific details to be identified by the designer for each individual building. The specific materials and components would be specified for an appropriate foundation envelope system, which meets the site conditions. It is recommended that site analysis be done in all areas where frequent foundation envelope failures are known to occur.
- An educational program organized through local Home Builders Association locals and chapters of the Building Officials Associations should be delivered in the areas where the highest rates of failure are occurring. Short seminars could be presented at builder/building official/private inspector regular scheduled meetings and hands on workshops should be held with builder/inspector/professional trainer teams to facilitate the learning of new procedures which will prevent foundation envelope failures.
- The structural performance of foundations is an important fundamental purpose of Building Codes. To ensure all Code users are aware of the performance guidelines that are developed, it is recommended the guidelines be referenced in the Appendix of the Code and supporting documents. Furthermore, it is recommended that Warranty

Programs make this information available to their registered builders at a discounted rate to reduce their risks of future payouts.

- To ensure the frequency and cost of foundation failures is reduced; it is recommended a survey of failures be undertaken within three years to assess the various measures, which have been implemented. The same successful survey methodology should be used. A revised checklist to characterize foundation failures should be developed and disseminated to interested parties.
- The cost of foundation failures is high and with increasing development on marginal lands and infill sites, it is expected problems will continue unless research is undertaken to develop superior and simpler approaches to foundation design and construction. There is a need for further research and guidance on isolation methods, which eliminate the characteristics of poor soils, and superior foundation systems, which are more resistant to frost damage.

8 References

Marshall, R., and Martin M., (1993). Revised 1994. Defect Prevention Research Project. Canada Mortgage and Housing Corporation/Ontario New Home Warranty Program

Marshall, R., and Morrison J., (1995). Impact Analysis of Geotechnical Defects. Ontario New Home Warranty Program

95 REDUCTION OF CRACKS IN PLASTER REVESTMENT

A. Müller and R.H. Romagna

Universidade Federal de Santa Catarina, Florianópolis, Brazil

Abstract

In Brazil, can easily be verified in building constructions various problems such as cracks, unglued parts, humidity problems. Thus, consequently the durability of the building is affected.

In structural masonry building constructions this situation can be more critical, since this type of walls are load bearing and internal stresses cause deformation. Factors affecting cracks formation at the initial shape are: materials characteristics and proportions of raw materials; water absorption by substrate and workmanship. This work deal with the optimum proportion of the plastering mortar. Known the characteristics of constituents materials (cement, lime and sand), the water absorption of substrate and the workmanship; it was made a attempt of find the best proportion of them, who will result in the smaller amount and dimensions of cracks. It was studied also, the model obtained by the correlations. The search has carried out using the material employed on the Florianópolis-SC region.

Keywords: cracking, drying shrinkage, plaster.

1 Introduction

Volumetric variation mortars happens because of the chemical shrinkage during hydration of the cement portland or for the humidity variations and temperature. During the application of the mortar, this loses water for the base and also for evaporation, caused by the solar incidence, speed of the air, temperature and relative humidity.

This property has fundamental effect on performance of the coating, because it affects the watertightly and the durability of the same. As a consequence of these dimensional variations, the coating can have its performance harmed by the occurrence of cracking. It is justifiable the adoption of this theme for the academic

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and scientific, given middle the great number of cases to it related, present in the general picture of pathologies of the constructions of the country. Even so, the shrinkage depends, in a lot, of the base to be covered and the rehearsals accomplished separately in the mortars, don't get to reconstitute all the complex phenomenons of the which elapse the drying shrinkage, becoming therefore a property of difficult quantification.

The shrinkage is a phenomenon that happens in the mortars in the initial shape.

This deformation type happens mainly because of the loss of water of the mortar for the atmosphere for evaporation, and for the base, for evaporation, in the application in common substract. The shrinkage in general develops during it catches it and after the hardening of the mortar, in normal conditions of exhibition to the air. The shrinkage that still happens with the mortar fresh and that can be measured in sample it is a subject of volumetric contraction of the material for the exit of the mixture water. In the hardened mortar, after the exit of the water free present in the capillary emptiness, the shrinkage is provoked by the loss of the water adsortion, that is, loss of the water that is physically stuck to the wall of the capillary emptiness of the paste. In severe special conditions of dried, studies on cement pastes also describe the shrinkage for the exit of water interlamelar, present among the layers of the structure of the silicate of moisturized calcium (C-S-H) (METHA & MONTEIRO, 1994). The restriction of the shrinkage provokes, already in the first hours, the appearing of traction efforts and, consequently, an internal state of tensions. In the free shrinkage, that is to say, in non applied sample to any base type, the attaché is responsible it puts that restriction. In the mortars applied in substrate as the bricks and the concrete, the restriction of the shrinkage also happens because of the own adherence of the mortar to the base.

The restriction of the drying shrinkage causes the appearing of traction tensions in the mortar, when this á applied to a base. A combination of the elasticity modulus (E) and of the deformation (ε) and for shrinkage it can generate a tension value ($\sigma = E \ . \ \varepsilon$) that when reaching the resistance to the traction of the mortar, it takes to the cracking.

It leaves of the dried retraction it is reversible. The hardened mortar can be moved in the sense contrary to the of the retraction when it subjects, for example, to the weting.

Studies of the deformation in mortar [DÉTRICHÉ, 1978] they drove to the accompaniment, also of the evolution of the mechanical properties that you/they condition its behavior with relationship to the cracking: deformation module and resistance to the traction [TAMIN, 1986]. The first represents the deformability of the material when subject to tensions and according the capacity to resist to the traction tensions.

In this presented and discussed healthy work the results of values in six lines different from mortar, with and without addition of polypropylene fibers, its mass variation and the tensions in them existent with the age. It is a discussion limited by the fact that the used methods allow rehearsals starting from the age of a day of the mortar, and not in the first hours. Besides, it was not also possible the use of methods that allowed the mensuration of the real tensions originated by the shrinkage of the sample.

2 Materials and methodology

Physical and chemical characteristics of the materials employed for producing the plaster: Common Portland cement; hydrated lime; polypropylene fiber are shown in tables 1 and 2. The sand was washed and dryed in heater (105 ± 5° C), its characteristics are shown in figure 1.

Table 1: Common Portland cement characterisitics.

Physical testing	Average	Sd
Especific gravity (g/cm^3) (NBR 6474)	3,12	
Fineness (% retained)		
- sieve n° 200	1,75	0,22
- sieve n° 325	9,01	1,15
Especific surface-Blaine (m^2/kg)	315	7,0
Time of set (min)		
- Begin	206	8,8
- End	305	verif
Expansibility (mm)	1,00	0,41
Compressive strength (Mpa)		
- 1 day	13,6	0,40
- 3 days	26,10	0,80
- 7 days	31,40	0,80
- 28 days	39,90	0,60
Chemical testing	(%)	(%)
Al_2O_3	4,31	0,08
SiO_2	19,81	0,26
Fe_2O_3	2,60	0,05
CaO	61,47	0,21
MgO	5,09	0,33
SO_3	2,51	0,07
Fire loss	3,26	0,20
Free CaO	1,78	0,11
Ins. R.	0,63	0,10
Alc. Eq.	0,67	0,03

Table 2: Hydrated lime characteristics

Physical testing	
Fineness (% retained)	
- sieve n° 30 (0,600 mm)	≤ 0,5
- sieve n° 200 (0,075 mm)	≤ 15,0
Estability (NBR 9205/85):	No cavitys or protuberance
	0,52
- water/lime factor, by mass (g./g.)	200
- consistence obtained in the paste (mm.)	
Plasticity (NBR 9206/86)	164
Water retentivity (NBR 9290/86)	73
- water/lime factor, by mass (g./g.)	0,820
- consistence obtained in the paste (mm)	208
- Water retentivity	≥ 70
Capacity of incorporation of sand (NBR 9207/85)	
- proportion (lime:sand), by mass	1,00:2,20
- water/lime, by mass (g/g)	0,714
- consistence obtained in mortar (mm)	206
- heigth of mortar no extruded (cm)	3,7
Chemical testing (NBR 7175/92)	
Humidity (%)	0,1
Fire loss (%)	20,0
Insoluble em cloridric acid (%)	5,0
Fe_2O_3+Al_2O_3 (%)	0,99
CaO (%)	40,0
MgO (%)	28,0
Anidrido sulfúrico (SO3) (%)	0,04
CO_2 (%)	≤ 15,0*
No hydrated oxides (%)	≤ 15,0
(CaO + MgO) (%)	≥ 88,0**

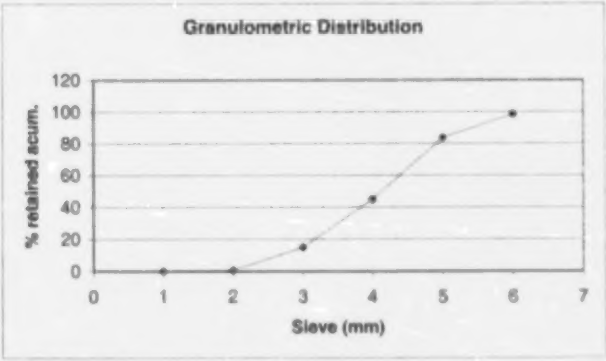

Fig. 1: Granulometric distribution of sand

Mass proportion of the 6 types of plaster used, as well as the percentage of addition of polypropylene fiber, are shown in tables 3 and 4.

Table 3: Plasters without polypropylene fiber

Plaster	Volume Proportioning Cement: Lime:Sand
A	1:1:6
B	1:2:9
C	1:3:10

Table 4: Plasters with addition of polypropylene fiber

Plaster	Volume Proportioning Cement:Lime:Sand	% Addition Fiber, by mass of cement
D	1:1:6	1.5
E	1:2:9	1.5
F	1:3:10	1.5

It was produced panels with 1,10m width, and 1,00m heigth, using concrete blocks, whose characteristics are shown in table 5.

Table 5: Concrete blocks characteristics

Dimensions (mm)			Absorption (%)	Specific Gravity (Kg/m^3)
W	H	L		
141	191	392	6.8	2008

Note: W = width; H = heigth and L = length

These panels were revested with the 6 types of plasters . For plasters A, B, and C the consistence, by flow table, was fixed in 290 ± 10 mm. For plasters D, E and F, the water/cement ratio was equal to A, B, and C plasters, respectly, and the consistence measured.

Deslocament measurement equipment (DEMEC GAUGE), with 0.002mm precision, were positioned as demonstraded in figure 2.

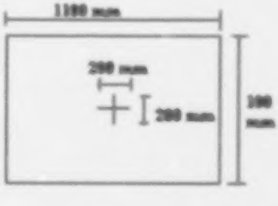

Fig. 2: Design of panel

Curing conditions were maintained constant, temperature varying 23 ± 2° C and 50 % air humidity. Measurements were done in 3, 7, 14, 21 and 28 days old.

3 Results and discussion

Table 6 shows the properties of the plasters used:

Table 6: Properties of plasters used

Plaster	Water / Cement	Consistence by Flow Table (mm) (NBR 13276, 1995)	Compressive Strength (MPa) (NBR 13279, 1995)	Absorption (%) (NBR 9779, 1995)
A	1.67	290.5	7.94	14.47
B	2.50	287.5	3.59	15.04
C	3.32	293.1	2.21	17.42
D	1.67	274.7	8.14	14.86
E	2.50	279.1	3.80	15.68
F	3.32	292.2	2.79	17.44

The consistence, by flow table, adopted for the plasters A, B and C was 290 ± 10 mm. For the plasters D, E and F, the consistence was measured. The results shows that, for a same percentage of addition of polypropylene fibers, the consistence, by flow table, reduces with the increment of cement in the proportioning of the constituent materials (cement:lime:sand). But the reduction of consistence do not reduce expressively the workability of the plaster.

Results obtained in testing shows that the compressive strength and absorption rises with the addition of polypropylene fibers; but the increment is not expressive.

Figures 3, 4 and 5 shows the comparison between the drying shrinkage of plasters with or without polypropylene fibers, for the 3 mass proportioning.

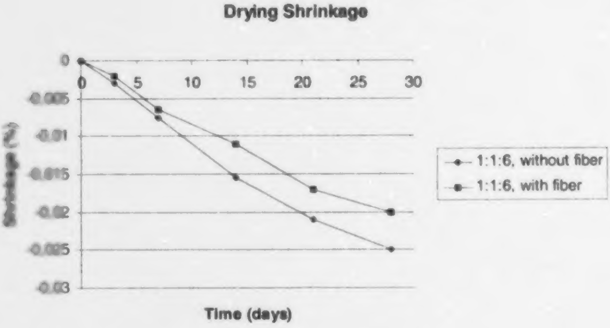

Fig. 3: Comparison between shrinkage of plasters A and D

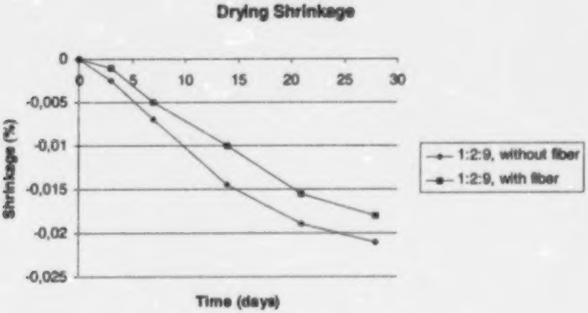

Fig. 4: Comparison between shrinkage of plasters B and E

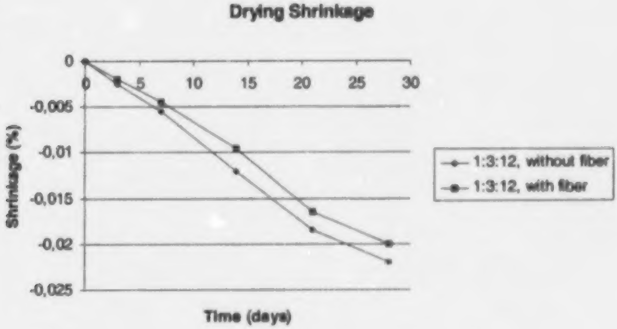

Fig. 5: Comparison between shrinkage of plasters C and F

For the 3 comparison of plasters with the same mass proportioning, can be noted that the addition of polypropylene fibers reduce significantly the drying shrinkage. For plasters more reach in cement (Plasters A and D) the reduction of shrinkage is more expressive, and can result in smaller tendency of cracks formation in walls revestment.

The polypropylene fibers improve the elastic properties of the plaster and, thus, reducing the drying shrinkage and the cracking of the revestment.

4 Conclusions

The addition of polypropylene fibers in plasters do not influence significantly critical properties of mortars, such as workability, compressive strength and water absorption, but can reduce the drying shrinkage. The reduction of shrinkage is more significant for plasters with more cement (mass proportioning 1:1:6 and 1:2:9).

Reduction of drying shrinkage in plasters can reduce the cracks formation, resulting in best quality and durability of parging, and, consequently, of the building.

Next paper will study other important properties of plasters, such as tensile strength, Young's modulus, and correlations with the drying shrinkage of plasters with addition of polipropilene fibers.

5 References

Associação Brasileira de Normas Técnicas – ABNT. Argamassa para assentamento de paredes e revestimento de paredes e tetos – Determinação do teor de água para obtenção do índice de consistência padrão. NBR 13276. São Paulo, (1995).

Associação Brasileira de Normas Técnicas – ABNT. Argamassa para assentamento de paredes e revestimento de paredes e tetos – Determinação da Resistência à Compressão. NBR 13279. São Paulo, (1995).

Associação Brasileira de Normas Técnicas – Argamassa e concreto endurecidos – Determinação da absorção de água por capilaridade. Método de ensaio. NBR 9779. ABNT, Rio de Janeiro, (1995).

Détriché, C. H. (1978) Analyse expérimentale du retrait de couches minces de mortier mesure depuis le moulage. *Matériaux et Constructions*, Vol. 11, No. 64, pp. 247-249.

Peled, A. and Guttman, H. (1992) Treatments of polypropylene fibers to optmize their reinforcing efficiency in cement composites. *Cement & Concrete Composites*, Vol. 14, pp. 277-285.

Tamin, P.F. (1986) Etude du comportament mecanique des revestments de façadesenduits. 138 p. *Thesis* (Docteur-ingenieur en Sciences et Techniques du Batiment) - Ecole Nationale des Ponts et Chaussées.

96 ELECTRICAL MEASUREMENT OF MOISTURE CONTENT IN POROUS BUILDING MATERIALS

P. Norberg

Royal Institute of Technology, Gävle, Sweden

Abstract

The moisture content (MC) of porous building materials is often estimated via measurement of the resistance between two electrodes inserted into the material. The most common material to which this technique is applicable is wood. In general, the MC reading obtained is affected by several internal and external factors, some of which are seldom taken into consideration in practice. The paper exemplifies some of the most important factors involved in the determination of MC. The background to the various expressions relating the electrical resistance to MC, temperature, etc., are reviewed particularly for wood.

Keywords: Electrical measurement, resistance, geometry, electrode configuration, moisture content, porous materials, wood

1 Introduction

Several common building materials are porous or even hygroscopic, e.g. solid wood and wood-based products, plasterboard, concrete, autoclaved aerated concrete, rendering, coatings, etc. The performance of these materials is in many respects related to their moisture content (MC). This quantity is defined as the ratio of the weight of water to the weight of dry material. Estimate of MC in wood is most often made by measuring the resistance between two electrodes driven into the material. In principle, the same technique can be used with almost any porous or hygroscopic material. Traditionally, however, wood has been the most typical material to which resistance methods have been applied. This is partly because wood is relatively soft and electrodes are fairly easy to install at an arbitrary depth. Another reason is that the variation in resistance between different wood species or different samples is much smaller than that caused, for example, by relatively small changes in MC.

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Various factors affect the measurement of resistance and they can be divided into at least three categories, namely, those involving:

- the size and geometry of the measured object in relation to that of the electrodes and also their configuration
- the interaction of the material (and the electrodes) with the surrounding climate
- the intrinsic properties of the material

The consequences of the first category are very often given by the instrument and electrodes used, but may to some extent be related to the skill of the operator. Of great importance in this context is also the understanding of the two concepts: resistance and resistivity. The second category involves environmental factors such as temperature and MC and the presence of thermal and moisture gradients in the material. The third category, finally, comprises e.g. the presence of conducting ions, the nature of the pore structure and the density of the material. In the end, the integrated actions of all these factors will affect the measured resistivity that in turn is frequently used for estimating the MC.

This paper presents the general principles and theoretical considerations for estimating, by resistance measurements, MC of porous and hygroscopic materials in general. Semi-empirical expressions will be shown that relate resistance to MC, temperature and some other factors that are specific to the materials considered. In particular, the effects of some of the factors mentioned will be exemplified using laboratory data obtained for wood.

2 Observations

A common observation made in the context of measuring MC in wood using pin- or nail-type electrodes is that the distance between the electrodes seems to be of very little importance to the obtained reading. It does not matter much if the electrodes are a few centimetres apart or if several meters separate them, provided that the MC is the same all over. The main reason for this result is that the resistance exerted by the bulk wood between the electrodes, even when mounted at very long distances from each other, is small compared with the resistance that arises closer to the electrodes. In Figure 1, R_{C_1} and R_{C_2} are associated with the wood/electrode interface and R_w with the volume resistance of wood (James 1994). However, it is not correct to consider R_{C_1} and R_{C_2} as regular contact resistances comparable to the situation when oxides are present on metallic surfaces. In this case, there is no physical barrier limiting the transfer of current but the resistance rather arises because of geometrical reasons, as will be further discussed below. In the presence of moisture gradients also large differences between R_{C_1} and R_{C_2} would be anticipated. The obtained moisture reading results from the total resistance in the electric circuit. Consequently, the electrode that is situated in the driest part of the wood will be decisive for the MC reading. This also means that the actual MC will always be underestimated when gradients are present.

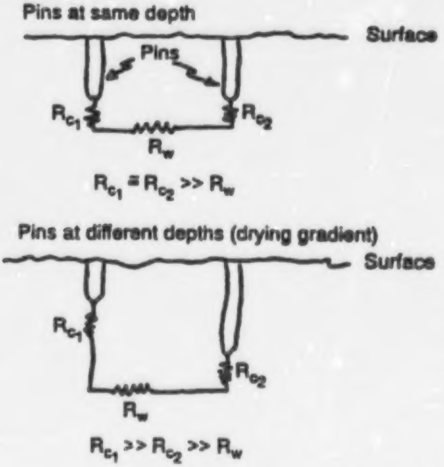
Pins at same depth
Surface
Pins
R_{c_1}
R_{c_2}
R_w
$R_{c_1} \cong R_{c_2} >> R_w$
Pins at different depths (drying gradient)
Surface
R_{c_1}
R_{c_2}
R_w
$R_{c_1} >> R_{c_2} >> R_w$

Fig. 1: Schematic showing the apparent distribution of electric resistance across two wood moisture electrodes (James 1994)

In the case of electric moisture meters, MC is related to the resistance or conductance obtained for the given electrode configuration. Normally, this is not a problem as the calibration is valid for the dedicated electrodes when inserted into a sample volume that may be considered infinite, i.e. a situation that may be illustrated in Figure 1. On the other hand, serious deviations in the estimates of MC may be expected if electrodes having different geometries are used with the same reading instrument without correcting for, primarily, the difference in cross-sectional area of the electrodes. Deviations are also expected to occur for essentially one- and two-dimensional wood samples or if the overall size of the sample is small, as a consequence of the narrowing of the electric field (Grahn 1997). In this case, the distance between the electrodes will also come into effect.

3 Theory

3.1 Resistance and resistivity

The resistivity ρ and conductivity σ are intrinsic material properties in contrast to the resistance R and conductance G, respectively, which are also dependent on the geometry of the conducting medium. It is essential for the understanding of the phenomena described previously to be aware of the difference between resistance and resistivity and their reciprocals, respectively. For a conducting medium with constant cross-sectional area A and length L, the resistance is:

$$R = \rho \frac{L}{A} = \frac{L}{\sigma A} = \frac{1}{G} \tag{1}$$

Equation 1 is applicable only if the cross-sectional area of the wood sample is the same as that of the electrode. The quantity L/A is generally referred to as the cell constant for the particular electrode configuration.

If the cross-sectional area varies along the length L, the resistance is given approximately by:

$$R = \rho \int_0^L \frac{dL}{A(L)} \tag{2}$$

Equation 2 reveals the effect of geometry of the conducting path on the resistance between the electrodes, as was discussed earlier. The dominating contribution to the integral (= the cell constant) would come from parts of the wood that involve small cross-sectional areas, i.e. the two wood/electrode interfaces when pin-type electrodes are used.

For more exact formulations of resistance, Laplace's equation must be solved subject to appropriate boundary conditions:

$$\Delta V(x, y, z) = 0 \tag{3}$$

where V is the electric potential. The influence of the actual geometry on the apparent resistance may also be studied by experiments using an electrolytic tank (Grahn 1997). If the electrolyte in the tank has the same size and geometry as the wood sample it may be used as a model for studying the effects of electrode configuration, sample thickness etc., on the distribution of the electric field and thereby the resistance across the electrodes.

3.2 Influence of temperature and moisture content

The decreasing resistivity of wood and other ionic conductors with increasing temperature was observed by Clark and Williams (1933):

$$\log \rho = A + \frac{B}{T} \tag{4}$$

where T is the absolute temperature and A and B constants at a constant MC. Equation 4 is essentially the logarithmic version of Arrhenius' well-known equation.

Another common relationship, originally proposed by Stamm (1927), expresses the effect of wood MC on resistivity at a constant temperature:

$$\log \rho = C + D \cdot \log M \tag{5}$$

where M is the moisture content and C and D constants at a constant temperature. This expression is utilised by most wood moisture meters and has been found to work satisfactorily within the typical range of MC where electric moisture meters are used, i.e. 7 - 30%.

The simultaneous effects of temperature and MC on the resistivity can be expressed in one single equation by superimposing Equations 4 and 5 (Norberg

1998a):

$$\log \rho = A^{*} + B^{*} \cdot \log M + \frac{C^{*}}{T} \qquad (6)$$

or from the practical point of view:

$$\log M = A + B \cdot \log \rho + \frac{C}{T} \qquad (7)$$

By taking the antilogarithm of Equation 7 the following expression is obtained:

$$M(\rho, T) = 10^{(A + B \cdot \log \rho + C/T)} = 10^{A} \cdot \rho^{B} \cdot 10^{C/T} \qquad (8)$$

Equations 6 - 8 are applicable to a sample of any porous material for which the resistivity is depending mainly on the moisture content. The validity of this expression must also be restricted to a certain moisture content interval where the moisture may be considered bound to the structure in a certain way. For wood, the fibre saturation point is often defined as the moisture content at which the cell walls are saturated with bound water with no free water in the lumens. Above this point, typically around 30% MC, the resistivity decreases much less with increasing MC than it does below, the fibre saturation point.

Many moisture meters are not equipped with facilities that enable direct compensation for temperature in the way suggested by Equation 8. Often the meter follows Equation 5 and the reading has to be manually corrected for the difference between the actual temperature and the calibration temperature. A useful relationship in this context may be obtained by taking the derivative of Equation 8 with respect to T:

$$\frac{\partial M}{\partial T} = -\frac{M \cdot \ln 10 \cdot C}{T^{2}} = -K_{T} \frac{M}{T^{2}} \qquad (9)$$

Equation 9 shows that the effect of temperature increases in the negative direction with increasing MC and with decreasing temperature. This is also illustrated in Figure 2 using limited data for European oak from the study by Norberg (1998b). As an example, assume that a moisture meter, calibrated at 20°C, results in an MC of 18% at a wood temperature of 10°C. The temperature correction estimated from Figure 2 at 18% MC and 10°C is -0.155% per °C or +1.5% MC for the -10°C difference, and thus the true MC is 19.5%.

When conducting measurements of resistance or resistivity in order to estimate the MC of e.g. wood, it is important to consider the possible risk that the electrodes and the temperature probe affect the conditions at the measuring spots by altering the temperature there. The conduction of heat to and from the measuring spots via the electrodes and the connecting leads should therefore be minimised by proper design and by keeping the size of heat conducting parts as small as possible. These precautions are particularly important to long-term measurements outdoors where the variations in temperature and radiation conditions are large over time.

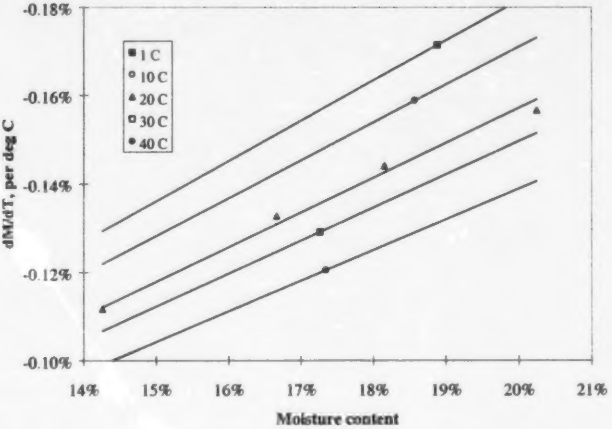

Fig. 2: The influence of MC on the apparent change in MC per degree change in temperature for European oak as measured with resistance-type moisture meter

3.3 Intrinsic properties

Very little work seems to exist regarding the effect of dry density on the resistivity of wood. The influence of density is generally considered weak but with a negative correlation with resistivity as reviewed by e.g. Keylwerth and Noack (1956), Lin (1967), and Vermaas (1982). Recently, on the other hand, the effect of dry density of wood was found to explain a great deal of the observed variance of a given data set (Norberg 1998b). An *ad hoc* adaptation was made by simply adding a linear term representing the dry density of wood to Equation 6, resulting in:

$$\log \rho = C_1 + C_2 \cdot \log M + \frac{C_3}{T} + C_4 \cdot d_{dw} \qquad \textbf{(10)}$$

A similar formula was also developed by Du et al. (1991), but the background work has never been published.

There are two reasons why the dry density should be considered in relation to resistivity. First, a denser material would imply more cell-wall substance per unit volume and, secondly, higher contents of extractives should also involve a higher concentration of conducting species in the cell walls. Further improvement of Equation 10, with regard to the density term, is possible by assuming a very simple structural model for wood, see Figure 3.

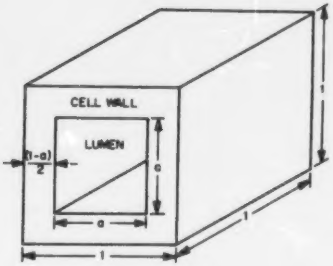

Fig. 3: Geometrical model for single wood cell (Siau 1984)

According to Siau (1984) the resisitivity of wood in the longitudinal direction is:

$$\rho = \frac{\rho^*}{1 - a^2} \tag{11}$$

where ρ is the longitudinal resistivity of bulk wood, ρ^* is the longitudinal resistivity of cell-wall substance and $1\text{-}a^2$ the fraction of cell-wall substance per unit volume of wood. Similarly, the dry density of bulk wood d_{dw} may be expressed in terms of the dry density of the cell-wall substance d^*_{dw}:

$$d_{dw} = d^*_{dw}\left(1 - a^2\right) \tag{12}$$

By combining Equations 11 and 12 the geometrical term can be eliminated and the following expression is obtained:

$$\rho = \frac{\rho^* \cdot d^*_{dw}}{d_{dw}} \tag{13}$$

If ρ^* and d^*_{dw} are considered constant then Equation 13 may be further simplified to:

$$\rho = \frac{C^*}{d_{dw}} \tag{14}$$

Equation 10 can be modified by substituting the *ad hoc* part of it, involving the dry density, with the expression given by Equation 14:

$$\log \rho = C_1 + C_2 \cdot \log M + \frac{C_3}{T} + C_4 \cdot \log d_{dw} \tag{15}$$

In order to conform to the particular experimental conditions prevailing in the study by Norberg (1998b), Equation 15 can also be written with MC as the dependent variable and substituting current i (nA) for resistivity:

$$\log M = A + B \cdot \log i + \frac{C}{T} + D \cdot \log d_{dw} \qquad \textbf{(16)}$$

A renewed regression analysis of the previous data using Equation 16 was made and the result is summarised in Tables 1 and 2.

Table 1: Result of regression analysis using Equation 16 on data for 12 specimens conditioned at 8 different combinations of relative humidity and temperature. Input data from Norberg (1998b)

Statistics	**Regression coefficients**			
	A	**B**	**C**	**D**
Coefficient	-1.08	0.0925	309	-0.298
Standard error of coefficient	0.0720	0.00421	15.3	0.0192
t-value	-15.0	22.0	20.1	-15.5

Table 2: Regression results for the total estimate in addition to the data given in Table 1

R^2	**Standard error of log M estimate**	**F-value**	**Degrees of freedom**
0.89	0.0161	241	92

The analysis presented in Tables 1 and 2 shows that Equation 16 provides a slightly better fit than did Equation 10 as given by Norberg (1998b). The significance of the coefficients and of the total regression equation can be tested according to t- and F-statistics, respectively. The critical t- and F-values tabulated for 92 degrees of freedom and an arbitrary significance level of 0.005 are 2.63 and 19.5, respectively. Comparison with the data in Tables 1 and 2 shows that all of the coefficients and the regression equation as such are highly significant.

4 Discussion and conclusions

The present paper discusses the various factors affecting measurement of MC of porous materials in general and of wood in particular. The cell constant for the electrode configuration applied influences all measurements of resistance in electrolytes. Many investigators, as it appears from the studied literature, neglect this important fact. Thus, to enable comparison between results originating from different types of equipment, estimates of MC in building materials should be correlated to resistivity rather than resistance.

The effects of excitation voltage, AC or DC, and its duration, on polarisation phenomena have not been considered in this paper. If the measurement of resistivity aims at estimating the MC, then making a rapid DC excitation during fractions of a millisecond with reversal of the polarity, should eliminate the possible effects of polarisation. In this way, any capacitance associated with the material/electrode interface would be eliminated and only the resistivity of the electrolyte in the material will contribute to the result.

Equation 7 relates the electrically determined MC to resistivity and temperature. This expression is generic to ionic conductors and should be applicable to most porous and hygroscopic materials, at least within a certain range of MC where the conduction of charges follows the same mechanism. Further improvement of Equation 7 in relation to moisture measurements in wood has been accomplished by taking into account the dry density of wood (Equation 16). This was done using a simple model of the basic wood-cell structure enabling the resistivity to be expressed as a function of the dry density.

Similar models, based on the actual pore structure, could be developed for other porous materials, e.g. concrete and autoclaved aerated concrete. An additional complication with cementitious materials, such as concrete, is the fact that the pozzolanic reactions are expected to continue indefinitely and gradually alter the pore structure and thereby the resistivity (Whittington et al. 1981; Hanson and Hanson 1983; Hughes et al. 1985). This would require a time dependent factor when describing the resistivity.

Recently, work has started that will eventually make possible a description of the pore structure of autoclaved aerated concrete and renderings in conjunction with resistance-type measurements of MC under field conditions (Kus and Norberg 1999).

5 References

Clark, JD and Williams, JW (1933) The electrical conductivity of commercial dielectrics and its variation with temperature. *J Phys Chem*, Vol 37, pp 119-131.

Du, QP, Geissen, A and Noack D (1991) Widerstandskennlinien einiger Handelshölzer und ihre Meßbarkeit bei der elektrischen Holzfeuchtemessung. *Holz Roh-Werkstoff*, Vol 49, pp 305-311.

Grahn, J (1997) Electrical measurement of moisture content in wood. Influence of specimen geometry and electrode configuration. M.Sc Thesis, *TRITA-BYMA* 1997:6 E, Royal Institute of Technology, Stockholm.

Hanson, ILH and Hanson, CM (1983) Electrical resistivity measurements of Portland cement based materials. *Cement and Concrete Research*, Vol 13, No 5, pp 675-683.

Hughes, BP, Soleit, AKO and Brierley, RW (1985) New technique for determining the electrical resistivity of concrete. *Magazine of Concrete Research*, Vol 37, No 133, pp 243-248.

James, WL (1994) Fundamentals of hand held moisture meters: An outline. *Proc ASTM Hand Held Moisture Meter Workshop*, Madison, WI, Forest Products Society, 5 May 1993, pp 13-16.

Keylwerth, R and Noack, D (1956) Über den Einfluß höherer Temperaturen auf die elektrische Holzfeuchtigkeitsmessung nach dem Widerstanosprinzip. *Holz Roh-Werkstoff*, Vol 14, No 5, pp 162-172.

Kus, H and Norberg, P (1999) Evaluation of the long-term performance of water repellants on rendered autoclaved aerated concrete. Submitted to *8th International Conference on Durability of Building Materials and Components*, May 30 - June 3, 1999, Vancouver, Canada.

Lin, RT (1967) Review of the electrical properties of wood and cellulose. *For Prod J*, Vol 17, No 7, pp 54-60.

Norberg, P (1998a) Monitoring of wood moisture content using the WETCORR method. Part 1: Background and theoretical considerations. Submitted to *Holz als Roh- und Werkstoff*.

Norberg, P (1998b) Monitoring of wood moisture content using the WETCORR method. Part 2: Calibration and validation. Submitted to *Holz als Roh- und Werkstoff*.

Siau, JF (1984) Transport processes in wood. (Springer series in wood science) *Springer-Verlag*, pp 132-150.

Stamm, AJ (1927) The electrical resistance of wood as a measure of its moisture content. *Ind Eng Chem*, Vol 19, No 9, pp 1021-1025.

Vermaas, HF (1982) D.C. resistance moisture meters for wood, Part I: Review of some fundamental considerations. *South Afr For J*, No 121, pp 88-92.

Whittington, HW, McCarter, J and Forde, MC (1981) The conduction of electricity through concrete. *Magazine of Concrete Research*, Vol 33, No 114, pp 48-60.

97 SURVEY OF THE DURABILITY OF THE FACADES OF 4000 DWELLINGS IN NORTHERN PORTUGAL

V. Peixoto De Freitas, M. Sousa, and V. Abrantes
University of Porto, Portugal

Abstract

A deficient design and execution of building façades, as well as the use of inadequate materials are the main causes of the occurrence of pathologies on the envelope of new buildings. A department of the Portuguese Government, IGAPHE, requested that the Faculty of Engineering of the University of Porto study the durability of the solutions adopted on the retrofitting of the façades about 4000 dwellings, located in 32 residential areas in the North of Portugal, especially from a technical and economical point of view. In the first phase, the study consisted in the characterisation of the construction details of the façades and in the characterisation of the technologies adopted in the retrofitting. In a second stage, each façade was analysed "in situ" in order to assess the retrofitting results a few years after it was applied, followed by the characterisation and classification of the main pathologies observed. The relation between the cost and the durability of the adopted solutions was equally assessed, as well as the analysis of the adequability of the technologies used in relation to the support on which it was applied. Finally, a methodology to be used in retrofitting projects was defined.

Keywords: Cost, durability, façades, pathologies, retrofitting, technologies

1 Introduction

1.1 The problem of retrofitting buildings

In Portugal, efforts have been made in the last few years to build new residential areas, whereas in contrast little effort has been made to retrofit the existing buildings, resulting in the decline and abandonment of some old areas.

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According to the Euroconstruc-daei statistics for construction works in 1995 (Cormon 1997), Portugal, when compared to the 18 West European countries, occupies the sixth place as far as investment in the building sector is concerned, but is in the last place when it comes to retrofitting/maintaining the buildings, representing only 6% of total investment. The remaining European countries, on the other hand, invest an average of over 30% of total construction in retrofitting. Under these circumstances, it is necessary to carry out a certain number of actions aiming at:

- Identifying the buildings' most common pathologies and systematising them;
- Becoming familiar with the new materials and methodologies used to retrofits the buildings, which are necessarily different to those used in new construction works;
- Determining the durability of the retrofitting technologies;
- Developing a methodology for retrofitting projects.

1.2 Purpose of the study

In this work, we have analysed the envelopes of 4000 dwellings built in the 70s and 80s and repaired between 1988 and 1997, with the aim of evaluating the behaviour of the retrofitting technologies adopted on the façades.

In the first stage, we characterised the solution adopted on the construction of the façades and the retrofitting technologies used. In a second stage, the pathologies observed, were described and classified after the retrofitting process had taken place. In a third stage, we characterised the durability and quality of the solutions adopted and a relation between the cost and the durability according to 1998 values was established, although it is complex to do it and often lacks objectivity. Lastly, we defined a methodology that should be used in the retrofitting projects.

2 Characterisation of the buildings

This study is based on the analysis of 32 residential areas, on a total of 4000 dwellings built between 1977 and 1982, located in the northern region of Portugal.

All the analysed buildings were the result of a low cost construction and all presented basically the same building characteristics, with the exception of three residential areas, which consisted of single houses.

The majority of the buildings have two, three or four floors, usually with stores on the ground floor. Some of these buildings have basements, normally used as parking or storage space.

The structural system adopted, in 47% of the buildings, is made of a structure of slabs and solid reinforced concrete walls. In the remaining cases, a traditional system (a structure made up of pillars, beams and slabs) was adopted.

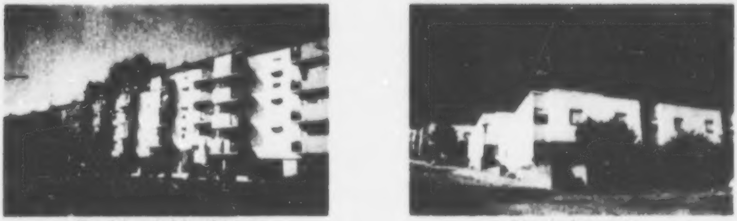

Residential buildings
Single houses

Fig. 1: Examples of examined buildings

3 Solutions used on the façades initial construction

The characterisation of the initial solutions adopted on the façades' composition allowed to assess the quality of the support and, at the same time, to analyse the adequacy of the retrofitting solution in relation to the support on it was applied.

The façades were cavity walls made of clay hollow brick masonry in 74% of the studied buildings, and made of cellular concrete blocks in the remaining cases.

The main types of external coverings used on the façades are indicated in Table 1.

Table 1: External coverings used on the façades - Initial solution

Ref.	External coverings used on the façades
1	Textured paint
2	Plastic paste on acrylic resins, with a textured
3	"Plastic" paint
4	"Epoxy" paint
5	Asbestos cement panels
6	Ceramic elements

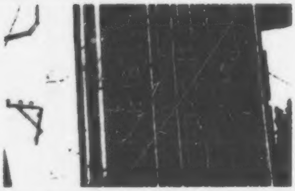
4 Solutions adopted in the retrofitting of the façades

In the 90s the façades were repaired according to the degree of degradation of the elements and to the costs that each solution implies. In Table 2 is described the different retrofitting technologies used.

Table 2: Technologies used in the retrofitting of the façades

Ref	Technologies used in the retrofitting of the façades	
1	External system composed by prefabricated elements mechanically fixed to a structure essentially composed by wood vertical elements attached to the support with metallic gadgets. An additional insulation is placed between the support and the covering panels, leaving an air space between the insulating material and the covering panels.	
2	External system of thermal insulation, made up of expanded polystyrene sections protected with a thin covering essentially made of resins, reinforced with fibreglass.	
3	Reinforced elastic paint	
4	Non-reinforced elastic paint	
5	"Textured" paint	

Table 2: Technologies used in the retrofitting of the façades

Ref	Technologies used in the retrofitting of the façades
6	Asbestos cement panels 

5 Observed pathologies on the façades after the retrofitting process

5.1 Façades pathologies

During the field studies carried out to each residential area in 1997 and 1998, several pathologies were observed on the façades of the retrofitted buildings, mainly due to defects in the design and construction and inadequacy of the retrofitting techniques used.

In Table 3 there is a description and classification of the main façade pathologies observed. It is important to mention that the retrofitting process took place between 1988 and 1997.

Table 3: Pathologies observed on the façades after having been restored

Ref.	Pathologies observed on the façades
P1	Cracked render thought the current surface of the façades
P2	Cracked render in over landing walls
P3	Cracked render under the window frame
P4	Microcracks on the external covering
P5	Degradation of the painting
P6	Dirtiness
P7	Stains on the façade due to micro-organisms
P8	Stains on the façades due to different thermal behaviour of the
P9	Insect cobwebs
P10	Efflorescences
P11	Degradation of the external thermal insulation system, specially at
P12	Degradation of the concrete surfaces

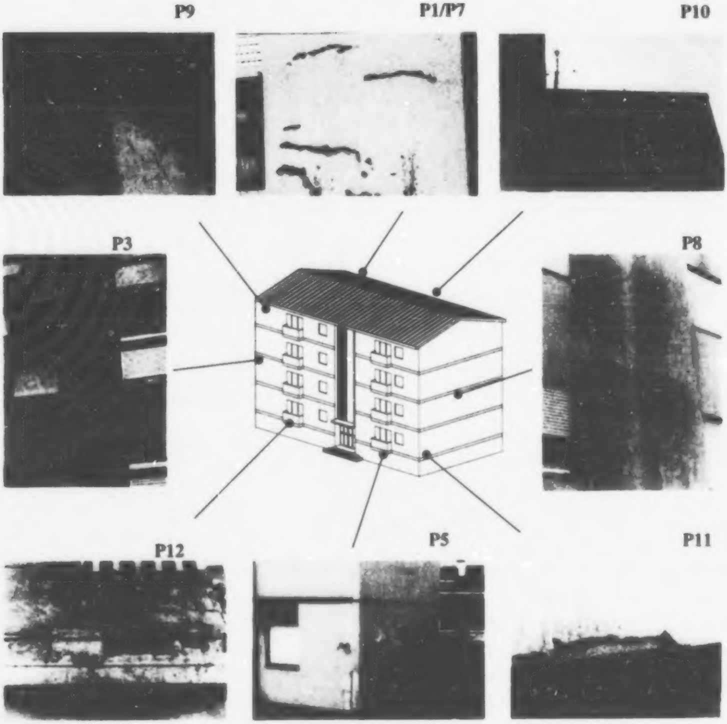


Fig. 2: Pathology examples

5.2 Description and classification of the degradation of the façades

The present state (1998) of each of the buildings' façades is classified according to the pathologies observed and the deteriorated areas, according to the criteria defined in Table 4.

Table 4: Classification of the pathologies according to the degraded areas

Level	Affected area
0	Difficult to assess
1	Occasional
2	Frequent
3	Generalised

The studied buildings were classified according to four levels of deterioration (D1 to D4), as presented in Table 5.

Table 5: Classifying the general degree of degradation of the buildings

Level	Degree of degradation
D1	No apparent degradation
D2	Slightly degraded
D3	Degraded
D4	Very degraded

6 Assessing the durability of the façades retrofitting techniques

6.1 The problem

It is important to analyse the durability and quality of the techniques used in retrofitting process of the façades of the 32 buildings studied. In order to carry out this analysis it is necessary to:

- Assess the quality of the support;
- Analyse the present state of the façades in relation to the time that has gone by since they were restored;
- Draw up a cost/durability of the retrofitting technologies model.

6.2 Assessing the quality of the support

The quality of the support influences greatly the good results of the restoring process. Therefore, it was classified according to three levels, depending on the type of the material used and the thickness of the outer leaves (Table 6).

Table 6: Evaluating the quality of the support

Level	Type of masonry	Thickness of the outer leaves
-1	Cellular concrete	< 15
0	Cellular concrete	≥ 15
	Clay brick	< 15
1	Clay brick	≥ 15

6.3 Assessing the present state of degradation of the façades in relation to the time since they were retrofitted

The buildings being studied were constructed in the 70s and 80s, and were retrofitted around 6 to 20 years later. In 1997 and 1998, each one of the 32 residential areas was visited and the pathologies observed were noted to analyse and classify the degree of preservation of the façades. Some of the façades had been restored over 8 years earlier and other was still undergoing retrofitting.

Observing the present state of the façades, we could notice that around 37% of the studied buildings were already degraded 6 years after they had been retrofitting, whereas 23% remained apparently untouched. The remaining buildings had been retrofitted less than 4 years before, thus being too early to assess a degree of deterioration.

Table 7: Classifying the residential areas according to the number of years since retrofitting

Level	Years following retrofitting
-1	0 to 3 years
0	4 to 5 years
+1	6 to 10 years

6.4 Durability model

The durability model presented in Table 8 makes it possible to analyse in qualitative terms the performance of the retrofitting technologies adopted, depending on the quality of the support (Table 6), the time that has gone by since the retrofitting was carried out (Table 7) and the buildings' present state of preservation.

Table 8: Durability model according to the retrofitting technologies adopted

Quality of the support	Time following retrofitting	Technologies used in the retrofitting of the façades					
		1	2	3	4	5	6*
-1	-1			D1/2	D1		
	0		D2/3	D2/3		D1	
	+1	D1/2	D3/4	D2/3	D2/3	D3	
0	-1			D2/3	D1	D1	
	0					D1	
	+1	D1/2	D3	D2 to D4	D4	D3 to D4	
+1	-1			D1	D1		
	0						
	+1		D2 to D4	D1/2 to	D2 to D4	D4	

(*) Information not available

By analysing the above-mentioned model, we may arrive at the following conclusions:

- The quality of the support has no direct influence on the performance of the retrofitting technology over the first 3 years;
- When more than five years have gone by, the façades reveal signs of degradation, except in some of the cases in which the support had a good quality and a type 2 or 3 retrofitting technology was applied (external thermal insulation system and reinforced elastic paint respectively);
- The external system composed by prefabricated elements (type 1) remains well preserved after a few years after it was applied, in spite of the composition of the support not being recommendable;

- The external thermal insulation system (type 2) responded in different ways, which means that the application method determines the performance;
- Reinforced and non-reinforced elastic paint (type 3 and 4 respectively) used on the façades will apparently preserve quite satisfactorily for the first 3 to 5 years;
- "Textured" paint (type 5) starts to deteriorate after 5 years, regardless of the type of support.

6.5 Evaluating the relation between the cost and the durability of the retrofitting technologies used on the façades

In Table 9 the durability of the retrofitting technologies used, based on the cost of each square meter of façade restored is characterised.

Table 9: Cost/durability model

Cost of the retrofitting	Technologies used in the retrofitting of façades					
	1	2	3	4	5	6
1-2			D1	D1 to		
2-3				D4	D3 to	D4
3-4		D2/3 to	D1/2 to	D2	D3	
4-5		D3/4	D1 to			
5-6			D4			
>6	D1/2	D1/2				

From this model following conclusions may be obtained:

- There is no uniform behaviour to the same retrofitting technique at approximately identical costs;
- The more expensive retrofitting methods respond more adequately.

In general terms, due to the diversity of performances, it may be said that each retrofitting process is a case on its own. However, it should not be forgotten that certain aspects as the application method of the technology and the specification of the technical characteristics of the materials used may influence the obtained results.

7 Retrofitting methodology

7.1 The approach

The retrofitting of already occupied buildings, in particular of the façades, requires a new approach of the different elements (the owner, the designer, the builders, the building managers and technical supervisors) and an excellent understanding of the process.

In Portugal, the methodology, which has been adopted, consists on the direct contact of the building companies to ask for an estimation of costs and usually is chosen the one with the lowest price. The time has come to change this principle, especially when we are dealing with high cost construction.

We should highlight the fact that most of the residential buildings in Portugal dating from the 60s and 70s are severely damaged and that the owners are all the inhabitants, making it difficult to come to a consensus of the different points of view when not founded on strong technical support.

7.2 The methodology

When a building presents a number of significant pathologies it is not possible to immediately start restoring without previously carrying out a study in different stages:

1. First the diagnostic study of the building's pathologies is drawn up, which should include possible repair solutions and an estimation of the costs;
2. A technical and economic analysis of the need to intervene and the owner's definition of the strategy which should be followed. Usually it depends strongly on the financial possibilities;
3. Once having defined the strategy, the retrofitting project should be drawn up and should include: a complete description of the retrofitting technologies adopted the correspondent costs and general and detailed drawings;
4. The building companies are asked to give proposals, which are then evaluated in technical and economic terms;
5. Finally, a team of technical and economic supervisors is hired to follow the process.

The following diagram presents a simple structuring of the several elements involved in the retrofitting process.

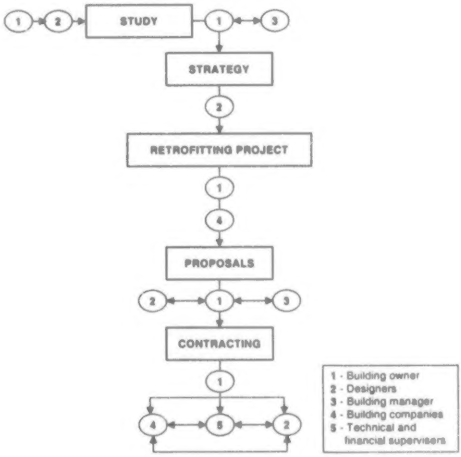

1
2
STUDY
1
3
STRATEGY
2
RETROFITTING PROJECT
1
4
PROPOSALS
2
1
3
CONTRACTING
1
4
5
2
1 - Building owner
2 - Designers
3 - Building manager
4 - Building companies
5 - Technical and financial supervisers

Fig. 3: Retrofitting process

In the more complex cases, especially when are dealing with large buildings, it may be justifiable to carry out a certain number of actions which will permit to test and validate the solution proposed, and simultaneously adjust the intervention cost after the diagnosis report has been presented.

8 Conclusions

The present study was the result of protocol signed between IGAPHE and FEUP and allowed a technical and economic analyses of the solutions used to build and retrofit 32 residential areas, a total of 4000 dwellings.

The initial typology and the retrofitting solutions adopted on the façades were characterised and compared with the pathologies observed, in order to organise them according to types and to classify the buildings' present conditions. It was also evaluated the relation between the cost and the durability of the retrofitting solutions adopted.

This study is intended to give an important contribution towards the issue of retrofitting buildings. It is almost certain that Portugal will require, in near future, a growing amount of investment in the maintenance and retrofitting of façades, just as in other European countries.

9 References

Abrantes, Vitor; Freitas, Vasco Peixoto de and Sousa, Marília (1998) *Reabilitação de Edifícios - Estudo do comportamento e análise tecnico-económica das soluções utilizadas nas obras de conservação e reabilitação de 32 empreendimentos do IGAPHE – Delegação Norte*, Protocolo de Cooperação IGAPHE/FEUP, Porto.

Cormon, Pierre. (1997) *L'évolution des technologies et des marchés dans les bâtiment*, UN- Miller Freeman, Division Construction, Paris, p. 16.

98 ASSESSING THE SERVICE LIFE OF BUILDING ENVELOPE CONSTRUCTIONS

C. Rudbeck

Technical University of Denmark, Lyngby, Denmark

Abstract

During the last 10 years, national standards have been developed in order to assess the expected service life of building materials and constructions and work is still progressing on the international level. Besides the current and upcoming standards, several methodologies have been developed or suggested at the national, international or individual level. The overview provided in this paper focuses on two two items: The first describes the current methods or standards established at a national level for either assessing and implementing service life requirements in the design stage of a building construction or for assessing the performance over time of building constructions in the building envelope. A review is provided that contrast the less practical against the more useful aspects of national and international standards. The second gives suggestions as to those methodologies that potentially can enhance parts of the upcoming standards. This objective here is to provide methodologies that are simple to use, since complex methods are often time consuming.

Keywords: building, building envelope, service life, standards, performance, life cost

1 Introduction

Durability has been an important aspect of the design process from the inception of the very first buildings. By observing the building constructions' performance over time, knowledge of best practice accumulated over the ages. New building designs

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were developed from trial-and-error, where faulty designs disappeared a long time ago whereas many of the buildings constructed by best practice are still found today.

In strong contrast to this, a large amount of the building stock constructed during the sixties suffers from serious defects due to changes in building design. In the post-war period the demand for housing was great and therefore one of the major issues was to build as many square meters as possible, while keeping the investment at a minimum. Buildings were expected to last 100 years, but 30-40 years after construction many of them are now in need of total renovation as the facades are deteriorated and the roofs are leaky.

Instead of focusing on the investment cost as the only economic parameter, life-cycle cost can also be used. Here, the costs of constructing, operating and maintaining a building is included in the calculations, making it economically sound to invest more money if the results are savings on operation or maintenance which justifies the larger investment. Life-cycle costs are treated in numerous publications, e.g. (ASTM 1994). Since life-cycle costs depend, among other factors, on service life and maintenance costs of the building, these should be estimated as accurately as possible. Such estimates should not only be given as an annual percentage value but also include information of the building components' and internal/external climate.

Methods for estimating service life have also received increased attention from building surveyors. Their customers want to postpone investments as long as possible (e.g. replacing a roof membrane) but also to have defects fixed before failure occurs (leaky roof). Faced with this dilemma the building surveyor must be able to give a good prediction of the remaining service life for the building component.

Currently no commonly available methodology exists for performing such assessment, making service life prediction a discipline that needs significantly more attention if it is to be used in e.g. life cycle cost calculations.

2 Current standards, guidelines and methods for assessing service life

Until now, a number of standards, guidelines and methods for assessing service life of building components have been developed at several levels.

2.1 National standards and guidelines

National guidelines have been produced in the last decade, including:

1. Japan - Principal Guide for Service Life Planning of Buildings (AIJ 1993)
2. Great Britain - Guide to Durability of Building and Building Elements, Products and Components (BSI 1992)
3. Canada - Standard S478: Guideline on Durability in Buildings (CSA 1995)

The Japanese guide states that service life should be predicted for the whole building, parts of the building or its elements, components or equipment. The end of the service life is either determined by physical deterioration or by obsolescence. Assessment of physical deterioration is based on information of the deterioration level at the end of the service life and the length of the service life. Based on these values, an annual physical deterioration, valid for one climate, can be calculated, helping in the assessment of the service life for future buildings. Calculation of the predicted service life under other stresses (level of use, climate etc.) and with other material qualities is performed using an equation including a combination of additions and multiplications with factors which describe the influence environmental agents, quality of work, quality of materials etc. have on the predicted service life.

The British guide recommends that service life is attained by reference to previous experience with a similar construction, measurements of the natural rate of deterioration combined with an assessment of the durability limit or results from accelerated tests which are performed on a scientific basis. As such methods can be imprecise it is recommended that more than one approach be used with a subsequent comparison of results. There is a lack of information regarding one method being superior to others. Another issue not being adressed is the possibility of calculating service life for a structure under one condition using information for other conditions.

From the beginning the Canadian guide drew on work done during the development of the BSI guide, but in due time evolved in width and depth. Unlike the two other guides, the Canadian guide also includes renovated buildings. Service life can be assessed by demonstrative effectiveness, modeling of the deterioration process and testing. Information is given regarding when to use which method and whether more than one method should be used. Like in the BSI Guide, the Canadian guide does not offer calculation routines for calculating service life.

2.2 International standards and guidelines

At the international level several organizations (ISO, CIB, RILEM, EOTA and ASTM) are working with the assessment of service life. Work with service life in ISO is conducted in ISO/TC59/SC14, "Design life of buildings" with the development of an international standard series entitled "Buildings - Service Life Planning" of which six parts are planned. The first part (ISO 1998) describes the general principles of the standard and outlines the methodology. The methodology for estimating service life follows the guide from AIJ (1993). In order to calculate the estimated service life of a component (ESLC), a reference service life (RSLC) is found for the component. The RSLC can be based on previous experience, building codes or test results, e.g. in accordance with the methodology developed by CIB-W80 and RILEM TC71-PSL and described in (Masters 1989) and further developed in (ISO 1998a). The ESLC can then be calculated by multiplying the RSLC by a number of modifying factors taking into account quality of materials, design, site work, indoor and outdoor environment, operating characteristics and maintenance level. This method is commonly known as the "Factor method". In the draft (ISO 1998) it is stressed that "double counting" should be avoided, i.e. that it must be ensured that the RSLC is not modified by any factors before using the method.

In CIB and RILEM research related to service life prediction has been conducted in CIB W80/RILEM TC-175 "Prediction of service life of building materials and components" and CIB W94 "Design for durability". A major objective for both commisions is to support ISO standardisation in the field. The result of the work in W80/TC-175 will be published in "Guide and Bibliography to Service Life and Durability Research for Building Materials and Components" (Jernberg 1997). Work in CIB W80 continues in four task groups named damage functions and environmental characterization, factorial methods, information technology and reliability and probabilistic methods. In CIB W94 the aim is to develop a design methodology to make it easy and natural for a designer to include durability in the design process. As problems may arise if communication between researchers and practitioners fails, the aim is also to produce guidelines for the presentation of research results in publications.

As accelerated testing is one of the methods for assessing the service life of building components and materials, ASTM developed a standard (ASTM 1996) to aid in the prediction of service life. Based on the ASTM standard a RILEM methodology (Masters 1989) was developed which was developed further into an ISO document (1998a). Such tests can be combined with deterministic or probabilistic analysis but this is only dealt with very superficially. An examination of a component begins with the characterization of the degradation mechanisms. Based on this, a list of degradations factors are compiled and pre-testing is conducted. These tests should demonstrate that rapid change in performance can be induced by exposure to extreme levels of factors on the compiled list. To ensure the validity of the the accelerated tests, other tests are performed under natural conditions, with a comparison of results. It is important to include the possibility of synergism, as the effects of a combination of weathering effects might be larger than the effects of the individual factors. Despite the test procedure which is outlined in the standard, no quantitative statement can be made on the precision of the testing, as the standard is insufficient to make a statistical analysis.

2.3 Individual or institutional methods for service life prediction

Research on service life prediction is also progressing at individual level or institutional level, with results published in international journals or at conferences. Different approaches have been followed in the research which is treated briefly here.

2.3.1 Structural approach

The aim of structural design is to calculate the strength needed of a construction in order to carry a mechanical load. In its simplest form the fracture stress of the construction should exceed the appearing stress by a certain factor (e.g. 3). In durability design a similar approach could be that the critical moisture content (above which damage is likely to occur) in a material should exceed the appearing moisture content by a certain factor. The analogy between structural design and durability design is mentioned by several, e.g. (Fagerlund 1996) and (Siemes 1996). However, deterioration is often due to a series of exceedings of an allowable stress level (e.g. moisture content) instead of just one instance. To include cycles of stresses in the calculations, S-N curves (S=stress, N=number of cycles) might be a solution as they are a common impact on a construction. A schematic representation of an S-N curve, assuming that damage is independent of the loading level, is shown in figure 1.

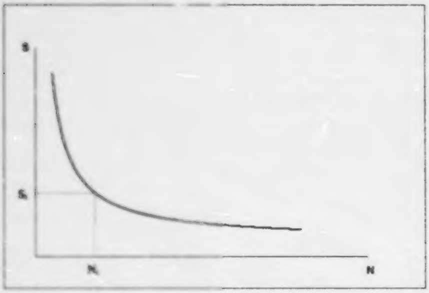

Fig. 1: Schematic S-N curve assuming damage being independent of load level

If the construction should last for N_i cycles, the load of each cycle should be kept below S_i. Other methods (Fatemi 1998) have been proposed to account for the relationship between damage and loading levels in structural engineering, but these are not treated here.

Another type of curve often used in structural design is the S-R curve shown on figure 2 where S denotes the characteristic value of the stress level and R denotes the load bearing capacity of the structure. When the stress level is higher than the load bearing capacity, the end of the constructions life is reached. Similar curves might be made in durability design, where S still would be the stress level, e.g. moisture content, and R would be the construction's performance level.

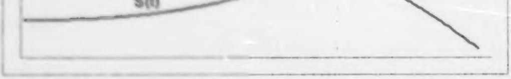

Fig. 2: Schematic S-R curve for a building component

2.3.2 Probabilistic approach

No two constructions follow the same performance curve or fail at the same time even though they are constructed and treated the same way. Material properties and quality of workmanship are statistically distributed and therefore probability analysis might be a useful tool in assessing the service life of constructions.

One way of using probability with regard to durability of building components is by including the probability of failure in economic calculations as suggested by Siemes (1985). Here the cost in case of failure of a component is included in the life cycle cost calculation. In equation (1) below T denotes life cycle cost, I is the investment, N is the design life of building, r is the interest rate, M_i is the maintenance cost in year i, $P\{F_i\}$ is the probability of failure in year i and D_i is the damage cost in consequence of failure in year i.

$$T = I + \sum_{i=1}^{N} \frac{M_i}{(1+r)^i} + \sum_{i=1}^{N} \frac{P\{F_i\} \cdot D_i}{(1+r)^i} \tag{1}$$

To solve equation (1) it is necessary to know the yearly probability of failure for the construction. This can sometimes be derived from experience if sufficient statistical data is available, but often the parameter is difficult to estimate. One way of estimating probability of failure is described by (Lounis et al. 1998).

Whereas Siemes operates with two states, either working or failure, (Lounis et al. 1998) uses a more differentiated view of the deterioration process. The method is under development for roofs, but it can be expanded to other parts of the building envelope. After a visual inspection, the condition of the roof is represented by a condition rating in the range of 1 to 7 corresponding to "failed" and "excellent" respectively. State 1 is an absorbing state (cannot be vacated unless repairs are made) whereas state 2 to 7 are transient states. For each roofing system and environmental condition, the probability of the roof going from one state to another during e.g. one year is estimated based on a categorization of visual observations of defects in the field. A certain roof could have a probability of 0.95 of going from state 7 to state 7 (unchanged condition) and a probability of 0.05 of going from state 7 to state 6. These values are combined into a matrix called the transition probability matrix. The advantage of the method is that only the present state of the roof is needed in order to predict the future performance of the roof. The disadvantage is that several probabilities have to be estimated before the method can be used and this is a time consuming and laborious task that requires a substantial number of field observations being collected using a standard procedure.

Siemes' method is not in itself a methodology for predicting service life, but use statistical data from other methods, such as that provided by (Lounis et al. 1998), as input to help a designer choose between two constructions.

3 Discussion of standards and guidelines

Work in the field of service life prediction is progressing along several major lines. Some methodologies focus on a deterministic approach, where figures are given as exact numbers without variation, whereas the methodologies focusing on a probabilistic approach include the effect of chance - or probability - in the values.

3.1 Factor method approach

The factor method, which is used as the tool of predicting service life in (ISO 1998) and (AIJ 1993) is based on very simple mathematics and is therefore very simple to use when the relevant factors have been decided. In the ISO guide the factors are just multiplied by the reference service life, while the Japanese guide operates with other simple mathematical operations i.e. addition, subtraction and division. An evaluation of the factor method by (Hovde 1998) indicated that although the method is simple, there are several issues which should be further evaluated in order to improve the use of the methods. These include the number of factors in the equation, span of factors, way of combining factors, relative importance and uncertainty of factors and the factor dependency of the material examined. When these issues are clarified, it should be noted that the predicted service life will be a number stating that the construction probably will last for a certain amount of time. However, there is always the uncertainty of something going wrong, e.g. due to poor workmanship or faulty design.

3.2 Structural approach

In structural engineering service life prediction has been performed in the last seventy years starting with development of linear damage rules as developed by Palmgren (1924). Much experience exists and the development of a methodology for building envelope construction would therefore create a seamless connection between structural and durability design. However, in order to use such an approach more information is needed. Functions describing the load and the resistance (the S and R functions in structural design which are schematically shown on figure 2) are needed for every type of environment, construction and material. To construct such functions huge amounts of data would be needed, making it a very cumbersome work.

3.3 Probabilistic approach

The advantage of the probabilistic approach is that it includes the aspect of uncertainty when predicting service life. With the inclusion of probability a building designer will have a tool helping to choose between e.g. a less expensive construction with a high probability of failure and a more expensive one with a low probability of failure. Probability belongs to the service life prediction as e.g. quality of materials and level of workmanship during construction are not fixed parameters, but change from one building site to another.

Two probabilistic approaches are sketched - one based on life cycle cost (Siemes 1985) that need input data for probability of failure for constructions, and one under development for roofing constructions (Lounis et al. 1998) where the construction is in one of seven states. The approach by Siemes suffers from having only two states and the construction will then change state from working to failed inside a short span of time unlike the approach by Lounis where progress of deterioration can be followed as the construction changes from one state to the other.

To implement these two approaches large amounts of data are needed. Siemes' method needs estimation of the probability of failure and the cost of these failures for different sorts of constructions throughout their lives, whereas Lounis' method needs transition probability matrices for each type of construction and climate.

4 Suggestions for improvement of standards and guidelines

To improve the usage of methods for service life prediction of building components, better knowledge outside research communities is needed, but before the methods are commonly accepted, some aspects should be dealt with. They can be divided into general aspects, aspects related to the deterministic approach and aspects related to the probabilistic approach.

4.1 General improvement

As more information regarding performance of building components is needed, both in the development of new methods and later on when the methods is to be widely used, there is a need for a systematic way of collecting and storing data.

Classification systems are also needed for building components and environments (indoor and outdoor climate). Groups of building components, with almost similar deterioration processes, should be formed to reduce the needed amount of data and similar approaches should be applied in description of the environment.

Finally, methods for predicting service life should be linked to the discipline of life-cycle cost. Buildings' service life is not calculated out of curiosity, but because the designer and owner would like to get the best performance per dollar. Being able to estimate a building's service life, the design of the construction can be optimized.

4.2 Improvement of deterministic methods

Regarding deterministic methods, as represented by the factor methods, some issues should be examined to facilitate their use. Calculation of predicted service life in (ISO 1998) only uses multiplication to combine the factors. It might be necessary to use other mathematical operations, expand the number of factors and add a possibility of weighing the different factors. However, care should be taken, as the method should be easy to use and still be able to give correct results.

Another improvement is that the method should be able to answer "How long can this construction last from now on?" at any time during the construction's life. If new information regarding the state of the construction is obtained it should influence the predicted service life.

Finally, better description of the method's use and better documentation of factors in the method are needed.

4.3 Improvement of probabilistic methods

To be useful in the prediction of service life, the probabilistic methods should be validated and strengthened by use of in-field data. When this has been carried out the method should, if possible, be expanded to other parts of the building envelope.

Other statistical methods than Markov chains used by (Lounis et al. 1998), should be examined for possible use in service life prediction.

5 Integrating durability in future building design

As mentioned, service life prediction should be combined with other disciplines to get the best result. Two major research projects are aiming at integrating, among other disciplines, service life prediction in the assessment of the building envelope.

The Integral Building Envelope Performance Assessment (IBEPA) project is aiming at developing a methodology to support the integral design and evaluation processes for building envelopes. The methodology include energy efficiency, durability, comfort, etc. Durability is important in the performance assessment, as determination of a building's predicted service life is crucial for other aspects (e.g. insulation thickness).

The Building Envelope Life Cycle Asset Management (BELCAM) project (Vanier and Lacasse 1996) was initiated to help asset managers in the decisions regarding when and how to repair their buildings. This is done by research in service life and durability, risk analysis and asset management. Current focus is on roofing, but the methodology is expected to be used for other parts of the building envelope.

Together, these two major research projects include the aspect of durability in a larger perspective as durability is not solved individually, but instead assessed together with other requirements, helping the building to perform at its optimum.

6 Conclusion

An overview of current methods and standards used for predicting service life of buildings is given. A review is provided that contrast the less practical against the more useful aspects of national and international standards. One improvement needed of the methods for predicting service life of building envelope components is to include life-cycle cost assessment. Furthermore a systematic method in collecting and storing data related to durability is needed. Deterministic methods, such as the factor method, suffer from lack of the method's use and description of the factors used. Such calculations should perhaps include more factors as well as weighing factors - an issue which should be examined in the future. However, an investifation of the basis on which these factors are derived should also be conducted. Probabilistic methods have been treated, and work is still needed in the form of in-field data to support the methods. These methods, potentially can be expanded to other parts of the building envelope.

Finally, there is a significant need for service life prediction given it is clearly an important part of the total assessment of building envelopes - both during the design and construction phase and later on, during use, where maintenance is needed.

7 References

AIJ (1993) *The English Edition of Principal Guide for Service Life Planning of Buildings.* Architectural Institute of Japan, Tokyo, Japan

ASTM (1994) *E917-94 Standard Practice for Measuring Life-Cycle Cost of Buildings and Building Systems.* American Society for Testing and Materials, PA, USA

ASTM (1996) *E632-82 Standard Practice for Developing Accelerated Tests to Aid Prediction of the Service Life of Building Components and Materials.* American Society for Testing and Materials, PA, USA

BSI (1992) *Guide to Durability of Buildings and Building Elements.* British Standards Institution. London, UK

CSA (1995) *Draft Standard S478-1995: Guide on Durability in Buildings. Canadian Standards Association,* Rexdale, Canada

Fagerlund, G. (1996) Moisture mechanics as a tool for service life prediction, in *Proceedings of the 7th International Conference on Durability of Building Materials and Components,* 1996, Stockholm, Sweden, 19-23 May, Vol 1, pp. 21-32

Fatemi, A., Yang, L. (1998) Cumulative fatigue damage and life prediction theories: a surver of the state of the art for homogeneous materials. *International Journal of Fatigue,* Vol. 20, No. 1, pp. 9-34

Hovde, P. (1998) Evaluation of the factor method to estimate the service life of building components, in *Proceedings of CIB World Building Congress 1998,* Gävle, Sweden, 7-12 June

ISO (1998) *Buildings - Service Life Planning - General Principles.* ISO/DIS 15686-1. International Standards Organisation, Geneva, Switzerland

ISO (1998a) *Buildings - Service Life Planning - Service Life Prediction Principles.* ISO/CD 15686-2: August 1998. International Standards Organisation, Geneva, Switzerland

Jernberg, P. (1997) *Minutes of CIB W80/RILEM TC 140-TSL/RILEM-SLM meeting in Capri, Italy, May 1997.* Conseil International du Bâtiment, Rotterdam, The Netherlands

Lounis, Z., Lacasse, M., Vanier, D. and Kyle, B. (1998) Towards standardization of service life prediction of roofing membranes, in *Roofing Research and Standards Development: 4th Volume,* ASTM STP 1349, Wallace, T. J. and Rossiter, W. J., American Society for Testing and Materials

Masters, W. L., Brandt, E. (1989) Systematic methodology for service life prediction of building materials and components. *Materials and Structures,* vol. 22, pp. 385-392

Palmgren, A. (1924) Die Lebensdauer von Kugellagern. *Verfahrenstechnik*, 68, pp. 339-341

Siemes, A. J. M. (1996) Framework for a procedure for design for durability, in *Proceedings of the 7th International Conference on Durability of Building Materials and Components*, 1996, Stockholm, Sweden, 19-23 May, Vol 2, pp. 1175-1184

Siemes, T., Vrouwenvelder, T. (1985) Durability - a probabilistic approach, *Durability of Building Materials*, vol. 3, no. 1, pp. 101-113

Vanier, D., Lacasse, M. (1996) BELCAM project: Service life, durability and asset management research, in *Proceedings of the 7th International Conference on Durability of Building Materials and Components*, 1996, Stockholm, Sweden, 19-23 May, Vol 2, pp. 848-856

99 IMPACT OF STUD GAUGE ON SERVICE LIFE — MVSS WALL

S.V. Thompson

Halsall Associates Limited, Toronto, ON, Canada

Abstract

The service life of exterior wall cladding should match or exceed the intended service life of the building. This study evaluated the impact of a substituted, lower gauge, steel stud on the service life of a masonry veneer on steel stud (MVSS) wall cladding system. The building walls at the subject building were modeled to estimate the "time-of-wetness". CSA S478-95 AGuideline on Durability in Buildings" and the related ISO standards were used to assess the corrosion rates and associated service lives of the steel studs in the expected environment. Our findings were that the substitution of 0.9mm thick steel studs for the specified 1.9mm studs will not reduce the theoretical service life of the studs to less than the design life of the building, providing the exterior wall is maintained to prevent leakage into the wall assembly. The heavier gauge steel only provides significant additional service life if the studs are exposed to a more corrosive environment, such as elevated moisture levels from the wall cavity.

Keywords: corrosion rate, durability, MVSS, brick masonry, steel stud, time-of-wetness, "MOIST", "FRAMEplus"

1 Introduction

The subject building consists of a ten storey high-rise office building located in St. Catherines, Ontario. The building was design-build, constructed in 1994. The building structure is primarily of cast-in-place reinforced concrete construction.

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The exterior walls typically consist of brick veneer with a steel stud back-up system. Semi-rigid glass fibre insulating sheathing insulates the steel studs from the cavity. The interior of the walls have foil faced drywall with glass fibre batt insulation between the studs.

During construction, the builder substituted a 0.9 mm thick steel stud for the specified 1.9mm stud. The purpose of this assessment was to determine the effect of the substitution of this lighter gauge stud on the predicted service life of the exterior envelope. Full analyses had been completed previously to verify that the studs were adequate structurally.

1.2 Wall construction

The available construction information indicates that the wall was constructed as follows:

- 100 mm face brick
- 25 mm air space
- 38 mm Glasclad AF530 insulation with seams taped and tie penetrations sealed with acoustic sealant
- Steel studs (0.91 mm)
- 150 mm of glass fibre batt insulation in the structural steel stud cavity
- Foil backed drywall with acoustic sealant in joints (over studs)
- Interior paint finish

The studs are understood to have been galvanized to meet the requirements of ASTM A446 Z275. This implies a zinc coating thickness of approximately 19.2 µm on each side of the stud.

Steel wire ties, electroplated with zinc, were used to secure brick to the steel stud back-up. Assessment of the durability of the ties and fasteners was outside the scope of the investigation.

The exterior envelope was constructed as a rain screen. Internal through-wall flashing membranes at the base of cavities direct the water outwards through drain holes. The drain holes also allow ventilation and some pressure equalization of the cavity, reducing the forces that drive water into the wall system.

1.3 Air barrier

At the time of construction, there was generally a clear understanding of the need for an air barrier within the envelope assembly. An air barrier is intended to provide a high degree of resistance to bulk air flow into or through the wall. Interior air, with its relatively high temperature and moisture content, can cause significant condensation formation within the wall if it is allowed to flow in contact with cold wall elements.

The interior drywall was designed to form the major component of the air barrier. The drywall was reportedly sealed around the window frames, to steel studs and at connections to the floor and ceiling. A peel and stick membrane was used at brace penetrations and other locations to make the air barrier continuous to the structure.

Air leakage testing, completed on the entire top floor, did not locate any substantial air leakage through the building envelope.

No air leakage paths through the building envelope are implicit in the design, and none were visually identified during review of the wall, including above the suspended ceiling.

Air leakage testing completed during construction confirmed that the envelope met the performance specification of 0.15 l/s/m^2. The test area found the air leakage to be 0.06 l/s/m2

A thermal scan of the walls was unable to detect any significant areas of concentrated air leakage at areas of brick cladding.

Based on the above information, it appears that the air barrier is effective.

1.4 Vapour retarder

The foil backing of the drywall provides adequate resistance to vapour flow through the walls. Vapour retarders are intended to minimize the static, one-dimensional flow of vapour through an assembly due to vapour pressure differences between the building interior and exterior. Minor discontinuities in the plane of the vapour retarder are not critical. Joints between boards are insignificant in width, and are typically located over studs, which provide substantially greater vapour resistance than the foil. Extra measures were undertaken during construction to improve the vapour retarder including sealant at board joints, but the continuity of these seals is inconsequential considering the diffusion flow of vapour. The air barrier, which restricts the three dimensional, bulk flow of air containing moisture is many times more critical in preventing condensation formation within the wall.

Based on the above information, it appears that the vapour retarder will serve its intended function.

1.5 Thermal insulation

The steel studs form a thermal bridge through the glass fibre batt insulation. The effects of this bridge are minimized by the presence of the semi- rigid insulation cladding, which passes over the studs. No severe thermal bridging was identified visually on site or by the thermal scan.

2 Analysis

2.1 Overview

To assess the effect of the stud thickness on the service life of the studs, we needed to first determine the service life of the zinc coating on the studs. The effect of the thinner studs will not become apparent until the zinc coating is consumed. The zinc coating thickness would be the same for both stud gauges. This information was confirmed with a major stud manufacturer (Bailey Metal Products) and two galvanizing companies (Dofasco and Pure Metal Galvanizing).

Two software packages were used to model the wall performance; "MOIST" developed by the National Institute of Standards and Technology, and "FRAMEplus" developed by Enermodal Engineering. MOIST was used to determine the "Time of Wetness" of the wall each year. MOIST is limited in that it provides a one dimensional analysis. It could either be run through a section of wall with insulation, or through a section of stud, but cannot, on its own, assess the impact of thermal bridging of the studs. To compensate for this inaccuracy, we completed a thermal analysis of the wall using FRAMEplus, which allows both the stud and the insulation to be modeled. The effective insulation resistance at the studs was determined and used in MOIST, by entering a lower R valve for the actual thickness of insulation.

The time of wetness was used to determine the rate of corrosion of the stud in the service environment. The critical wetness conditions were found to occur at the interface between the stud and the interior drywall, mainly during summer conditions, with high exterior temperatures and humidities relative to the interior air conditioned space.

The corrosion rates were estimated from various publications and the ISO standard. The service life of the studs was then estimated based on these corrosion rates.

2.2 Building design service life

Using CSA S478, we determined that the building, as an office building, was intended to have a design service life in the "long life" category of between 50 to 99 years.

2.3 Steady-state modeling - FRAMEPLUS

The brick veneer wall with steel stud back-up was modeled using one dimensional heat flow. FRAMEplus was used to determine the effect of thermal bridging through the steel studs. The effective thermal resistance at the location of the steel studs was determined from the temperature profile.

The wall was checked using steady-state vapour flow calculations under design winter conditions. According to these calculations, the wall would not experience an accumulation of condensation. The drying potential of the wall is conducive to minimizing the corrosion of metal components.

2.4 Non-steady modeling – MOIST

2.4.1 Climate Comparison

Climate data was assessed to determine the most appropriate data for use in MOIST. Hourly weather data provided with MOIST is provided by ASHRAE and covers one calendar year. St Catherines data is not included with MOIST, so other cities were reviewed to determine the closest match.

Table 1: Comparison of weather data for similar cities

	St. Catherines, ON	Detroit, MI	Toronto, ON
January, T2.5%	-16°	-16°	-18°
January, T1%	-18°	-18°	-20°
July, T_{dry}	30°	31°	31°
July, T_{wet}	23°	24°	23°
Degree days	3664	3659	3646
15min rainfall	23mm	28mm	25mm
1 day rainfall	77mm	78mm	121mm
Annual precipitation	807mm	849mm	801mm

As seen from the table, the design temperatures for July indicate the Detroit is warmer in the summer. The use of Detroit data then is a conservative, since the risk of wetness in the summer (due to the change in direction of vapour flow) is greater than would be expected in St. Catherine's. Also, higher rainfall is another conservative assumption. Although Toronto is geographically closer, Detroit is climatically a better match, and was selected for the analyses.

2.4.2 Time of wetness

Time of wetness includes all times when the relative humidity is greater than 80% and the temperature is greater than 0°C

MOIST describes the behaviour of the wall in response to changing environmental conditions by modeling heat and moisture flow. The program is a one-dimensional model and the version used allows for the coupling of inner layers of the wall to indoor and/or outdoor air. It also allows for the condensation of vapour and capillary transport. The program does not allow for the formation of ice, but this is not predicted to occur in this wall.

Modeling with MOIST used the effective insulating value at the steel studs as the area of interest.

The time period modeled was two years. The first year of the model was included to minimize the effect of assumptions regarding initial moisture contents. The results from the second year modeled were used to determine the time of wetness for the wall studs.

The time of wetness varied for different orientations, with south being the worst case. . The time of wetness at the outer edge of the stud (glass fibre) was consistently less than 250 hours per year falling in the t1 (<10 hours per year) and t2 (10 to 250 hours per year) categories of CSA 478-95. The time of wetness at the inner edge of the stud (gypsum wall board) was consistently less than 850, falling at the low end of the t3 (250 to 2500 hours per year) category. A sensitivity analysis was completed varying assumed conditions on the south elevation. This is summarized in Table 2. The time of wetness varied between 626 and 838 hours per year. This variation has no significant effect on the predicted corrosion rate or predicted service life that can be calculated.

Table 2: Time of wetness (hours per year) for changes to various parameters

Time of Wetness (hours per year)	Cavity Ventilation Rate			
	50 ACU		6 ACU	
Scenario:	Gypsum wall board	Glass fibre	gypsum wall board	glass fibre
North	793	219	611	0
East	796	209	682	0
West			670	0
South:				
Base Scenario	838	246	647	0
Outside Node	698	56	823	0
Interior Film Coeff. =24			626	0
Exterior Film Coeff. =24			670	0
Absorptance of brick =.65			639	0
Absorptance of brick =.80			670	0
Permeance Paint =320			640	0
Permeance Paint =29			646	0
Permeance Brick =138			679	0

Actual air changes per hour will be a function of temperature and wind. Stack effect and wind were considered and air changes are predicted to vary with both 6 and 50 being reasonable under different environmental conditions.

2.5 Prediction of service life

The calculated time of wetness is classified by CAN/CSA-S478-95 as a category of t2 to t3 which is typical of indoor air without climatic control. The building is located in an urban area typified by a rating of P2 (deposition of SO_2 of 35 to 80 $mg/m^2/day$), but the steel studs are sheltered within the building envelope. Thus an exposure rating of P1 (<35 $mg/m^2/day$) was used to indicate the level of pollution due to sulphur dioxide (SO_2). Similarly, a rating of S1 (deposition of chloride $<$ $60 mg/m^2/day$) was used to indicate the level of contamination due to salt spray since the outside environment would be classified as S2 (60 to 300 $mg/m^2/day$) and the building envelope again provides sheltering from contamination.

Service life predictions were then made using the corrosion rates identified in the International Standards ISO 9223:1992 and ISO 9224:1992. The Canadian Durability Standard, CAN/CSA-S478-95, is similar to the ISO standards, with the difference being the greater scope of coverage and the inclusion of corrosion rates in the ISO standards.

A rating of t3 was assigned for time of wetness. Combining these ratings with the other exposure ratings of S1 and P1, noted above, an overall corrosivity category of C3 (medium) was assigned for the zinc, and C2(low) for the steel. Using the following corrosion rates and thicknesses, the service lives were predicted:

Table 3: Service life prediction

Element	Thickness (mm)	Corrosion Rate			Time to Fully Consume (years)
			First 10 years (mm/year)	Beyond 10 years (mm/year)	
Rates in Predicted Environment					
Zinc	39 mm	C3	0.9	0.9	43
Steel (Provided)	0.91mm	C2	5	1.5	585
Steel (Specified)	1.91mm	C2	5	1.5	1250
Rates in Most Corrosive Environment					
Zinc	39 mm	C5	7	7	5.6
Steel (Provided)	0.91mm	C5	65	60	14
Steel (Specified)	1.91mm	C5	65	60	31

Category C5 (very high corrosivity) was calculated to demonstrate the effects of poor maintenance. Category C5 would be achieved with a time-of-wetness of more than 5500 hours (or 60% of the year).

3 Conclusions

Our calculations predict the life expectancy of the zinc coating on the stud to be approximately 45 years. The life expectancy of the 0.91mm stud is predicted at about 20% section loss, or 117 years, beyond the design service life of the building. The additional years that could be provided by the specified additional 1 mm of steel become irrelevant.

Masonry veneer, steel stud walls can be prone to premature deterioration. From these calculations, it is evident that the durability of the wall is not so much dependent on the construction specifications, but on the maintenance of the building, which should keep the environment within the wall as designed. If the envelope is not maintained, and the interior of the wall is allowed to become wet for a large portion of the year, the service life to 20% loss of section would be dramatically reduced, to as low as eight years. This is the type of failure mechanism familiar to many of us who have dissassembled leaking wall assemblies, revealing severly corroded studs, early in the life of a building.

4 References

Enermodal Engineering. (1995). *FRAMEplus*, Version 4.0. Waterloo, Ontario. (software)

Douglas M. Burch, Building and Fire Research Laboratory, National Institute of Standards and Technology. (1997) *MOIST*. NIST, Washington, D.C. (software)

Canadian Standards Association. (1995) *Guideline on Durability in Buildings*. CSA, Etobicoke. CSA S478-95.

International Standards Organization. (1992) *Corrosion of Metals and Alloys – Corrosivity of Atmospheres – Classification*. ISO, Geneve, Switzerland. ISO 9223.

International Standards Organization. (1992) *Corrosion of Metals and Alloys – Corrosivity of Atmospheres – Guilding Values for the Corrosivity Categories*. ISO, Geneve, Switzerland. ISO 9224.

American Galvanizing Association. (1994) *Hot Dipped Galvanizing for Corrosion Protection of Steel Products*. AGA, Washington, D.C.

National Association of Corrosion Engineers. (1980) NACE Corrosion Engineer's Reference Book, *Longer Lasting Protection – How Galvanizing Works*. NACE, Houston, Texas.

American Society of Heating, Refrigeration and Air Conditioning Engineers. (1989) *ASHRAE Fundamentals Handbook*, Section 28. Atlanta, GA.

ASM International. (1987) *Metal Handbook Ninth Edition, Volume 13, Corrosion*. Metals ark, Ohio.

American Standards for Testing Material. (1993) *Specification for Weldness Carbon Steel Chain*. ASTM, Philadelphia, PA. ASTM A466/A466M-M93.

American Standards for Testing Material. (1994) *Specification for Stainless Steel, Electrolytic Zinc-Coated, for Light Coated Mass Applications*. ASTM, Philadelphia, PA. ASTM A591/A591M-M89 (1994).

100 INVESTIGATION ON THE BEHAVIOUR OF POLYMER-CEMENT COMPOSITES IN ETICS RENDERINGS

F. Vallee,[1] N. Blanchard,[2] M. Rubaud,[1] and A. Gandini[3]

[1]Centre Scientifique et Technique du Bâtiment, St-Martin D'Hères, [2]Centre Scientifique et Technique du Bâtiment, Marne-la-Vallée, [3]École Française de Papeterie et des Industries Graphiques (INPG), St-Martin d'Hères, France

Abstract

The study of the ageing of polymer / cement composites under hygrothermal strains may lead to increasing the durability of these materials. The loss of flexibility observed on the composite during the accelerated ageing phases can not be attributed to a degradation of the polymer in aggressive concrete environments. Nevertheless, post-hydration of the initially poorly hydrated cement induces the development of a rigid inorganic matrix within the composite that becomes less flexible.

Keywords: polymer–cement, external thermal insulation, durability, ageing, microstructure, weathering.

1 Introduction

External Thermal Insulation Composite Systems (ETICS) are complex systems that are principally used to cover walls with an insulating material (e.g. expanded polystyrene), mechanically or adhesively anchored to the substrate. On this insulating layer are applied a base coat, containing a glass fibre mesh, and two other coats, namely the base and finishing coats (see Fig. 1). The present study deals with the base coat (rendering) which is a 3-mm thick polymer–cement composite. This rendering undergoes very high thermal and hygric strains that can lead to cracking. Our aim is to develop a better understanding of the origin of these problems.

Indeed the purpose of this study is two-fold:

1. To investigate the processes responsible for the changes in the properties of the materials, linked to the alteration of their structure induced by artificial weathering (cyclic temperature and/or moisture variations);
2. To establish correlation between these features and the appearance of cracking in order to propose measures to improve the durability of the initial satisfactory properties.

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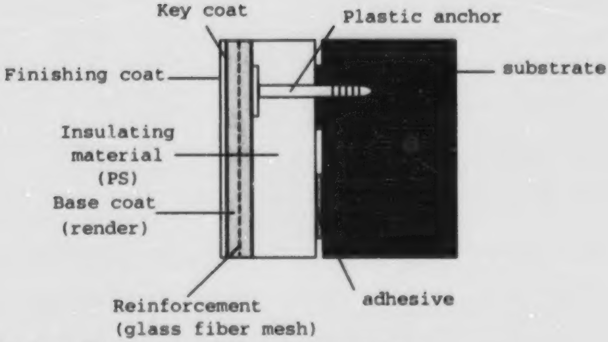


Fig. 1: External thermal insulation composite system

2 Experimental methods

2.1 Materials

Five latex-based materials were used in this investigation. They included copolymers of: vinyl acetate/vinyl versatate (VA/VV), ethylene/vinyl Acetate (EVA), styrene/acrylate (SA) and acrylic monomers (PMMA/Butyl Acrylate). The properties of these latex materials are given in Table 1.

These emulsions were used to prepare the polymer–cement composites that are identified using the same lettering as the corresponding latex material. These renderings are manufactured products with particularly complex formulations. All of them are bi-component systems: the (B) rendering is a powder (made of cement, fillers and additives) requiring the addition of an extra binder (polymer + water) whereas the other ones are pastes (fillers, polymer, water, additives) requiring addition of cement. Table 2 gives the essential components of their formulation.

Table 1: Properties of latex and polymers

Property	VA/VV (A)	EVA1 (B)	EVA2 (C)	SA (D)	PMMA/BA (E)
Appearance	White, milky	White, milky	White, milky	White, milky	White, milky
Solid content (%)	50 ± 1	54 – 56.5	50 ± 1	50 ± 1	50 ± 1
Density (g/cm^3)	1.06		1.09	1.04	
pH of latex	5	4 – 5	4.5	7.5 – 9	8.5 - 10
Particle size (μm)	0.6	0.5 - 2	0.7	0.1	0.15
Tg	10	6	5	22	15
MFFT*	3	0	4	20	14
Emulsifying system	Anionic / non anionic	Polyvinyl Alcohol	Cellulosic derivative	Anionic	

*Minimum Film Formation Temperature

Table 2: Basic formulations of composites

Components	(A)	(B)	(C)	(D)	(E)
Cement (%)	23	16	20	23	23
Polymer (%)	7.3	5.0	7.6	4.6	6.9
Mineral fillers	SiO_2, $CaCO_3$	SiO_2, $CaCO_3$	SiO_2, $CaCO_3$	SiO_2, $CaCO_3$	SiO_2, $CaCO_3$

* Except for the B composite, for which a CPA CEMI 42.5 cement was used, the cement was CPJ CEMII B 32.5.

2.2 Samples preparation and determination of mechanical properties

2.2.1 Polymer film preparation

Films of polymer were prepared by drying a thin layer of latex at 23°C and 50% RH (23/50) for 7 days ("initial samples"). Some of these films were kept for 7 days or 1 month in different aggressive environments, namely: (i) in water (23°C), (ii) in a pH 13 solution and (iii) in an oven at 70°C ("aged samples"). These treatments were followed by a drying treatment (at 23/50) lasting 7 days.

2.2.2 Composite preparation

After mixing the components, 3mm thick specimens of polymer–cement composite were prepared and cured for 28 days at 23/50 ("initial samples"). The ageing process consisted, on the one hand, of immersing some specimens in water for 7 days at room temperature, and on the other hand, in submitting others to 140 hygrothermal cycles. Each cycle was composed of 3 hours at 70°C followed by 3 hours water spraying at 23°C. Specimens were then dried for 7 days at 23/50 before testing.

Mechanical properties were evaluated by tensile tests (v=2-mm/min on samples 3-mm thick for the composites and 50mm/min on samples 0.25-mm thick for the polymers) under 23/50 conditions at the initial and aged stages.

2.3 Analysis

The evolution of polymeric structures was followed by FTIR, ^{1}H-NMR, and thermal analysis (TGA/DTA and DSC). The results were compared to those obtained on the corresponding polymers Soxhlet-extracted from the composite, using methylene chloride as a solvent. The microstructure of the composites was investigated by Scanning Electron Microscopy on polished surfaces using the backscattering electron technique, and X-Ray Diffraction (XRD).

3 Results and discussion

3.1 Mechanical properties of composites

Tensile tests showed that the composites, whatever their nature (Powder + Extra binder or Pastes + Cement) and their formulation, lost their flexibility after the ageing phases (see Figures 2 and 3).

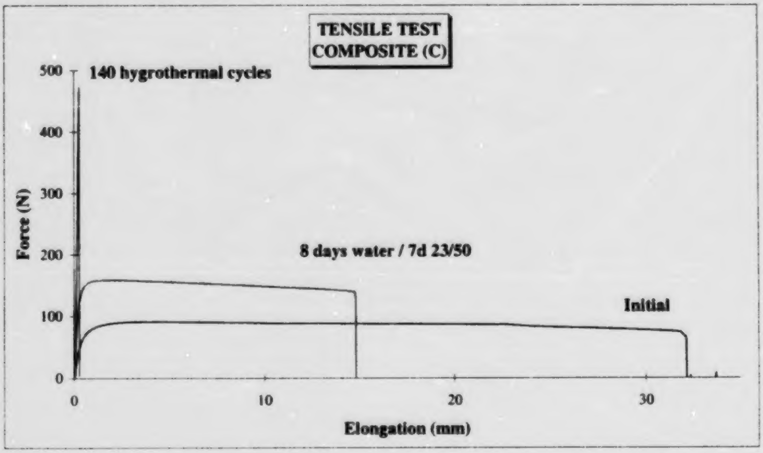


Fig. 2: Tensile tests of the composite C at initial and aged stages

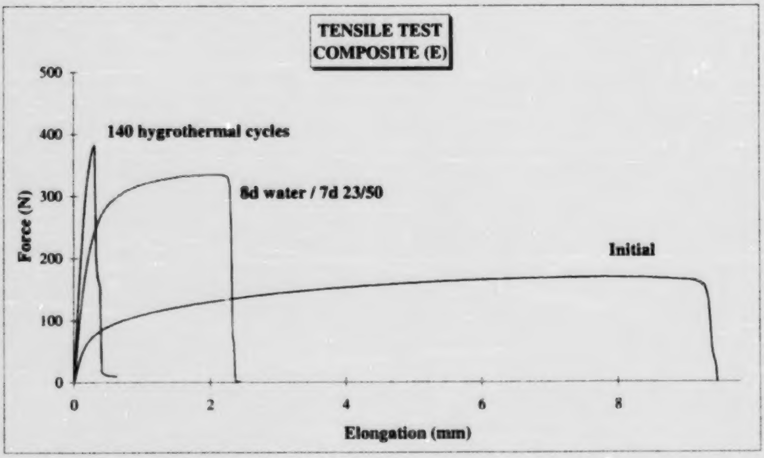


Fig. 3: Tensile tests of the composite E at initial and aged stages

Three hypotheses can be proposed to explain this phenomenon: (i) a modification of the organic phase; (ii) an evolution of the mineral matrix, or; (iii) polymer/cement interactions.

Polymer films were subjected to several aggressive environments in order to evaluate their stability in the inorganic phase.

3.2 Mechanical properties of the polymers

Three aggressive agents were deemed able to influence the structure and thus the mechanical properties of the polymers:

1. water (which can lead to a solubilisation, a lixiviation or a re-emulsification of the polymer);
2. temperature (which can lead to a plasticiser extraction or act as a catalyst for the crosslinking of some polymers), and;
3. pH because the cement medium is a particularly basic environment (pH = 13/13.5 during hydration) that can promote the (partial) hydrolysis of some polymers.

Figure 4 shows that water does not influence the mechanical behaviour of the tested polymers. Moreover, analyses of the immersion waters showed no trace of polymer. This proves that neither solubilisation nor re-emulsification of the polymer occurs. Nevertheless, results indicated that organic components were extracted from the film during the immersion phases.

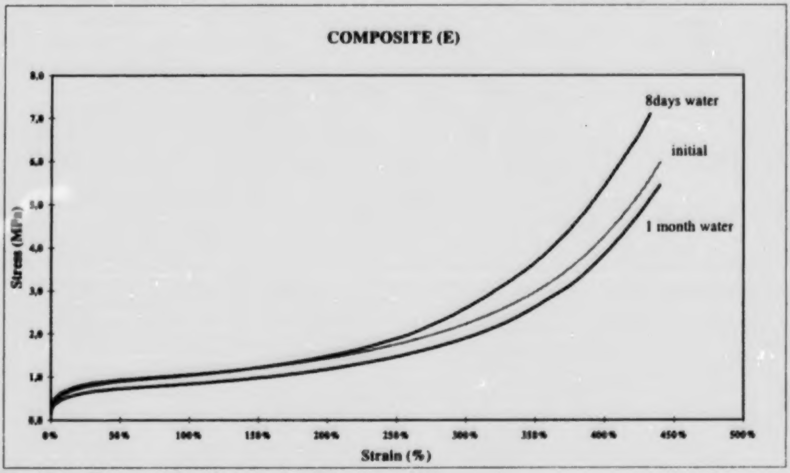


Fig. 4: Influence of water on the (E) composite

The IR spectrum of the product extracted from the EVA1 showed the characteristic peaks of polyvinyl alcohol with residual acetate functions, suggesting the lixiviation of stabilising additives.

In more general terms, colloids and surfactants present in the film, because of their particularly high hydrophilic character, can in fact be removed from the film during the immersion phases, but with no consequence on the residual polymer mechanical properties.

It was also found that temperature has no negative influence in this context but rather seemed to bring about a minor positive action since the polymers gained better stress and strain at break after one month in the oven (see Figure 5).

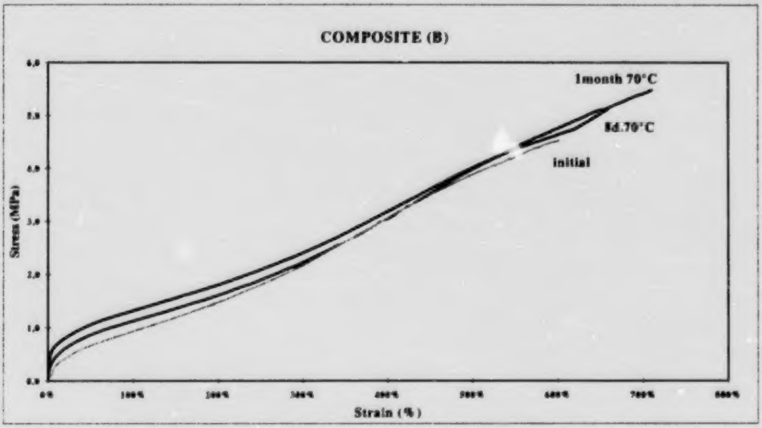


Fig. 5: Influence of temperature on the (B) composite

Conversely, pH seemed to cause a slight loss of flexibility of the polymers (see Figure 6) which may be explained by a partial hydrolysis, as indeed observed on the IR spectra. Table 3 shows the evolution of the optical density for the O-H and C=O bands.

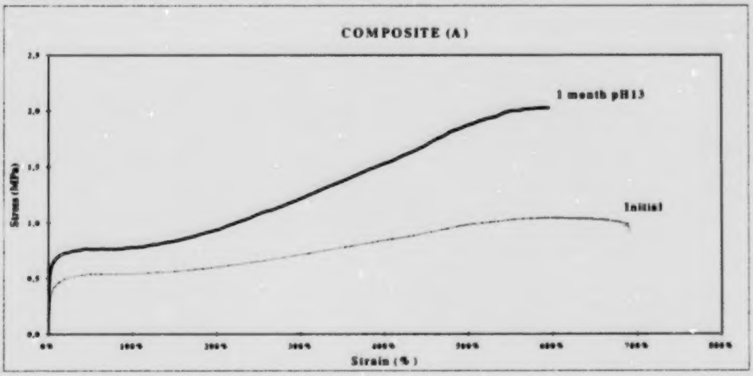


Fig. 6: Influence of pH on the (A) composite

Table 3: Absolute and relative optical densities of the OH and C=O bands for the EVA2 polymer at the initial stage and after 7 days in a pH 13 solution at 70°C

	Optical Densities				
EVA stage	OH (3444 cm^{-1})	C=O (1735 cm^{-1})	CH2 (1435 cm^{-1})	OH/CH2	C=O/CH2
EVA2 initial	0.17	0.77	0.46	0.37	1.67
EVA2 7days pH13/70°C	0.35	0.61	0.45	0.78	1.35

This set of results, which were in fact common to most of the numerous systems studied, shows that the polymers used here are well adapted for an application with cement such as ETI.

Nevertheless, in order to check the pertinence of these results, analyses of polymers extracted from the composite were also carried out. The Soxhlet extraction method is well suited for this kind of study. Figure 7 shows the IR spectra of both the EVA1 polymer film and of the products extracted from the initial and aged (B) composites.

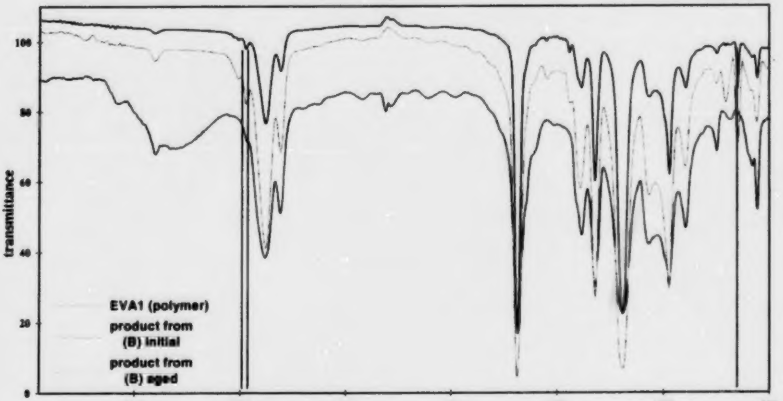


Fig. 7: IR spectra of the EVA1 polymer film and of the products extracted from the initial and aged (B) composites

The strict correspondence of the peaks tends to prove that no structural changes occurred during the stay of the polymer in the cement environment. However, a number of new peaks, characteristic of C=C bonding, can be observed on the spectra of the extracted products. The corresponding ^{1}H-NMR spectra showed that these peaks appeared because of the presence of an organic additive (coalescing agent), also removed from the composite during the Soxhlet extraction, which indeed contains at least one aromatic ring in its structure. As a consequence, the polymer extracted did not suffer any damage that could explain the loss of flexibility of the composite. Nevertheless, even if the nature of the removed polymer is the same, Table 4 shows a clear decrease in the extraction yields as the materials were undergoing the ageing treatment.

Table 4: Soxhlet extraction yields (%) for the polymers and the initial and aged composite

	Polymer	Initial composite	Aged composite
(A)	90	61	33
(B)	36		
(C)	78	64	44
(D)	25	17	13
(E)	97	43	25

Two hypotheses can help explain this trend, namely: (i) a change in the polymer chemical structure, or (ii) the establishment of polymer/cement interactions.

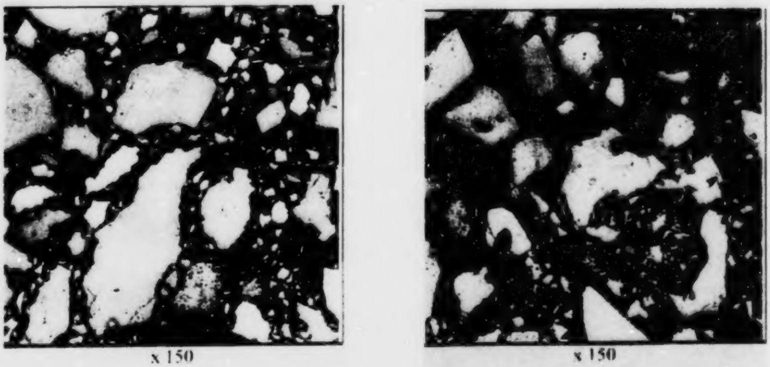
In the former case, an increase in polarity, induced by the (partial) hydrolysis of the polymers (polyvinyl acetate into polyvinyl alcohol or polyacrylate into polycarboxylate), would decrease the solubility of the polymer in methylene chloride, the low-polarity organic solvent used for the extractions. At the same time, this chemical change would introduce a stiffening effect on the polymer chains associated with the polar or ionic groups formed.

In the latter case, the formation of organic-inorganic complexes, insensitive to methylene chloride, could also cause the decrease in the extraction yields and a loss of flexibility of the composite. The inorganic moieties could be responsible of interchain interactions which, as a consequence, would result in higher stiffness and lower solubility in organic solvents.

3.3 Microstructure

The analysis of backscattering electron images of polished surfaces provides a means whereby the microstructure of cements and concretes may be characterised. Indeed, the analysis in a backscattered mode is based on the grey level differences among the phases, which, in turn, depends on the actual chemical composition of each phase.

The SEM observations of the composites allowed to reveal three kinds of contrasts, viz. large grey zones corresponding to the mineral fillers, black zones corresponding to porous areas and white zones, whose sizes were variable (but in any case below 100μm), corresponding to anhydrous cement particles.



x 150

x 150

Fig. 8: Microstructure (x 150) of the (C) composite after 28 days of curing at 23/50 (left) and after the 140 hygrothermal cycles (right)

The micrographs of the samples at the initial stage revealed a great number of anhydrous cement particles within the matrix after 28 days of drying at 23/50. This clearly reflects a low hydration rate (see Figure 8-left-). The proportion of anhydrous particles was found to be much lower for samples that underwent the 140 hygrothermal cycles (Figure 8-right-). Moreover, the observations at higher magnification showed that a "hydration ring" which was visualised by a greyish band of cement hydrates surrounded each remaining anhydrous particle. These observations suggest that the cement, initially with poor hydration, becomes re-hydrated during the accelerated ageing phases as it comes again in contact with water.

The use of Image Analysis provided an estimate of the amount of anhydrous cement, determined in terms of surface coverage. This gave about 7% after the 28 first days of curing which decreased to only 2% after the ageing process.

These results were confirmed by XRD analysis. Diffractograms of samples taken at the initial stage (Figure 9-left-) showed peaks of relatively high intensities corresponding to bi- and tri-calcic silicates (C_2S et C_3S) at $2\theta = 32\text{-}33°, 34.5°, 41\text{-}42°$ and 52° and tri-calcic aluminates (C_3A) at 33.2° and 59.3° which are the three main components of anhydrous cement. In certain cases, small peaks attributed to gypsum, used in cement as a regulator set, could also be observed ($2\theta = 11.5°$ and 20.8°). This is corroborating evidence for the very early stoppage of hydration.

The peaks of anhydrous cement were much lower on the diffractograms of aged samples (Figure 9-right-). The gypsum peaks had disappeared and small peaks of cement hydrates, particularly ettringite, became noticeable ($2\theta = 9°$ and 15.8°).

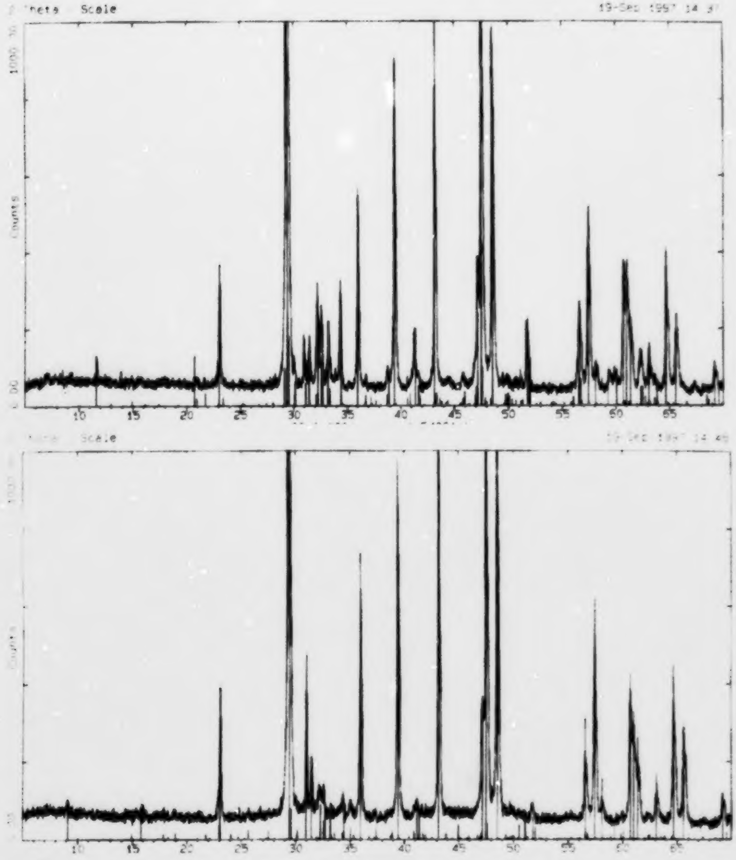


Fig. 9: Diffractograms of the initial (top) and aged (bottom) (C) composite

These results showed very important modifications of the cement matrix. At the initial stage, plasters are composed of an inorganic matrix whose poor cohesion can be explained by a low content of hydrated cement. In this configuration, the polymer brings an essential contribution to the mechanical properties of the composite whose flexibility is linked to the Tg of the macromolecules and the polymer/cement ratio. During the accelerated ageing phases, the cement comes again in contact with water and the development of cement hydrates reinforces the inorganic matrix. As a consequence, the composite looses part of its flexibility. Indeed, re-hydration causes, on the one hand, an increase in pH which can lead to a higher hydrolysis of the polymers, and, on the other hand, the release of new calcium ions which can interact with the polar groups on the polymer chains as often described in literature (Chandra et al. 1981; Chandra and Florin 1987; Ben-Dor et al. 1985)

4 Conclusion

The present state of this study enabled us to propose preliminary suggestions concerning the explanation related to the complex features associated with the ageing of the polymer/cement composites. However, a critical question remains: why is there such a low extent of cement hydration after the 28 first days of curing? In order to find a reasonable answer, further work will be conducted to study the influence of curing conditions on drying. Indeed, the 3-mm thick composites may not have enough time to get a good hydration because water may (partly) evaporate before being able to hydrate the cement grains. Based on available literature, the influence of the type and characteristics of the latex (i.e. monomer, Tg, particle size, emulsifying system) on cement hydration is also a logical way to pursue this investigation. Work is in progress to explore both avenues.

5 References

CHANDRA, S., BERNTSSON, L. and FLORIN P.: (1981) Behaviour of calcium hydroxyde with Styrene-Methacrylate polymer dispersion, *Cement and Concrete Research*, Vol.11, N°1, pp. 125-129

CHANDRA, S. and FLORIN P.: (1987) Interactions of polymers and organic admixtures on Portland cement hydration, *Cement and Concrete Research*, Vol.17, pp. 875-890

BEN-DOR, L., HEITNER-WIRGUIN, C. and DIAB, H.: (1985) The effect of ionic polymers on the hydration of C_3S, *Cement and Concrete Research*, Vol.15, pp. 681-686

PART SEVEN

ROOFING MATERIALS, COMPONENTS AND ROOF MAINTENANCE MANAGEMENT

ROOFING MATERIALS AND COMPONENTS

101 COMPARATIVE TESTING AND RATING OF THIRTEEN THERMOPLASTIC SINGLE PLY ROOFING MEMBRANES

C.G. Cash

Simpson Gumpertz & Heger Inc., Arlington, MA, USA

Abstract

We tested samples of poly[vinyl chloride] (PVC), thermoplastic olefin (TPO), and other thermoplastic single ply roofing membranes using a battery of physical tests before and after heat aging and accelerated weathering tests. We cut all test from commercial products purchased on the open market.

We ranked all test results in descending order from the product with the most advantageous property (equaling 100%) to the least advantaged. The average of the test percentage ratings on each product were then ranked from the highest rating to the lowest rating.

We used this procedure previously to develop data for the preparation of ASTM consensus Standard D 6221, *Standard Specification for Reinforced Bituminous Flashing Sheets for Roofing and Waterproofing*.

Keywords: ABC, accelerated weathering, EP, EVA, performance ranking, physical testing, PVC, roofing, single ply, thermoplastic membranes, TPO.

1 Introduction

Product or system selection can be difficult for the designer who desires to use a thermoplastic single ply roofing systen, because eight manufacturers offer thirty PVC (poly[vinyl chloride]) single ply roofing membranes and fourteen manufacturers offer forty-four "Other Prefabricated Sheet-Applied Membranes" according to the most recent NRCA materials guide (NRCA 1998). Two of these manufacturers offer both a PVC and an "other" membrane.

In the absence of an sufficiently long history of successful performance, there are very few ways a designer can objectively select a system for use. Some possibilities include:

- comparison of physical test data compared to appropriate standards, or
- comparison of physical test data obtained from testing competing materials,
- or reliance on data obtaned from accelerated exposure.

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We examine each of these three selection methods in this paper and contrast the advantages and disadvantages of each technique. Our data can also be used as a base line to compare the data obtained by testing other systems, as long as the same test methods are used.

2 Sample selection

We cut all of our test specimens from the inner convolutions of commercial products purchased on the open market. We tested specimens from one roll of each product. Because of the uniformity we expect in these factory made products, we feel that the specimens we obtained are a true representation of the products selected.

We would have preferred testing samples of all the available products, but economics required that we limit our study. We feel fortunate to have obtained thirteen different products for testing, and only two of these were from the same manufacturer. We obtained membranes based on the following polymers: eight rolls of PVC, two rolls of TPO (thermoplastic polyolefin), one roll of ABC (acrylonitrile butadiene), one roll of EP (ethylene-propylene), and one roll of PVC/EVA (poly[vinyl chloride] - ethylene vinyl acetate alloy).

We cut all the samples after the plastic sheets had been conditioned to equilibrium in the controlled temperature and humidity room of our laboratory. This room maintains a constant 50% relative humidity and a temperature of 23°C (73°F).

3 Test methods

We used all the test methods required by ASTM D4434, except the puncture resistance tests. We do not currently own the puncture resistance equipment required. In addition, we performed some tests suggested by a manufacturer that are not now ASTM standard tests.

3.1 ASTM test methods

We used the following ASTM test methods in this study:

- D570 Standard Test for Water Absorption of Plastics
- D638 Standard Test Method for the Tensile Properties of Plastics [using D412 die C - 64 mm (2½ in.) jaw gap, 0.85 mm/s (2 in./ min.)]
- D751 Standard Test Methods for Coated Fabrics
  Grab Tensile [76 mm (3 in.) jaw gap, 5 mm/s (12 in./min.)]
  Thickness [dial caliper]
- D1204 Standard Test Method for Linear Dimensional Changes of Nonrigid Thermoplastic Sheeting or Film at Elevated Temperatures [6 h at 80°C (176°F)]
- D2136 Standard Specification for Coated Fabrics - Low Temperature Bend Test [after 4 h at -40°C (-40°F)]
- D3045 Standard Practice for Heat Aging of Plastics Without Load [56 days at 80°C (176°F)
- G53 Standard Practice for Operating Light and Water-Exposure Apparatus (Fluorescent UV Condensation Type) for Exposure of Nonmetallic Materials

3.2 Other test methods

We used the following non-ASTM test methods:

3.2.1 Sheet mass

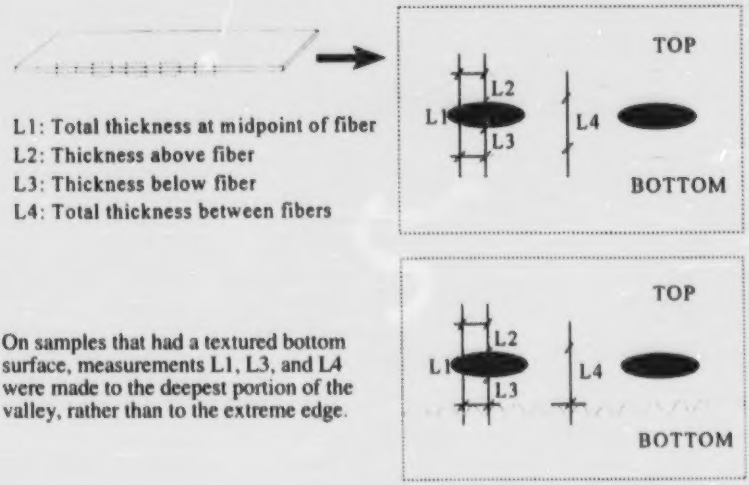
We die cut specimen squares 45 mm (1.78 in.) on a side with a razor knife, and weighed each square using an analytical balance. Ten times the average mass in grams equals the mass in pounds per 100 square feet. Multiply the mass in pounds per 100 square feet by 48.82 to convert to grams per square metre.

3.2.2 Fabric mass

We extracted the polymer compound from the specimens in 3.2.1 with a micro extractor using THF (tetra-hydrofurane or tetra methylene oxide) as the solvent, and dried and weighed the reinforcing fabric recovered. We did not report the mass when the polymer extraction was incomplete.

3.2.3 Optical thickness

We photographed five vertical sections of each sample in the machine direction (the length) and the cross machine direction (the width) using a 1.6 X objective lens and a 1.6 X zoom setting on the microscope. We made the four measurements shown in Figure 1 in each photograph.



L1: Total thickness at midpoint of fiber
L2: Thickness above fiber
L3: Thickness below fiber
L4: Total thickness between fibers

On samples that had a textured bottom surface, measurements L1, L3, and L4 were made to the deepest portion of the valley, rather than to the extreme edge.

Fig. 1: Location of optical thickness measurements

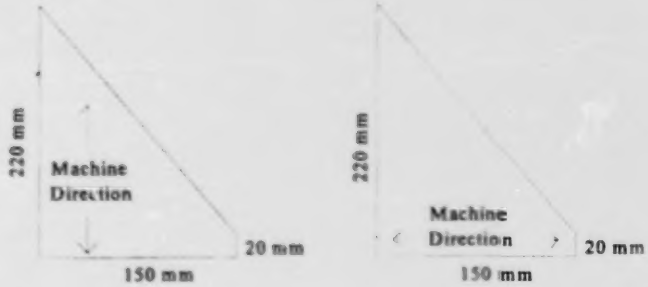


Fig. 2: Wicking test trapezoids

3.2.4 Wicking test

We cut the trapezoids shown in Figure 2 from each sample, and hung the lowest 20 mm in a water - methylene blue bath for 24 hours. We sectioned each specimen to measure the height of the wicking observed

3.2.5 Seam strength

We tested the strength of laboratory prepared heat welded 100 mm (4 in.) wide seams using the grab tensile test. We recorded that the seam strength was greater than the tensile strength if the sheet failed outside the lap area.

4 Results and discussion

These test data and our observations on samples placed in a ultraviolet condensing relative humidity apparatus are the basis for the following discussion and our conclusions.

4.1 Comparing test data to standards

Table 1 lists our test data on PVC membranes and the requirements of ASTM D4434. Table 2 lists our test data on membranes based on "other" polymers and the proposed ASTM requirements for TPO membranes. It is very difficult to select the "best" membrane from these data because different grades of the same product have different test requirements that are not comparable, most notably in the area of tensile properties.

4.2 Rating data

Table 3 lists all the data for samples 1 through 13. We rated these data by setting the maximum value obtained in each test to a rating 100, calculating the rating for each of the other products on a proportional basis. Where the "best" physical test result is zero, we set the rating at 100 for the test value zero and a rating of 100 for the maximum value obtained. These ratings are shown in Table 4. The ratings for each sample are averaged to measure the relative over-all rating of each product. In this series of tests the products average rating ranged from a high of 82 points to a low of 52 points.

4.3 Accelerated weathering

To the date of writing this paper, we have logged 3,000 hours in our ultra-violet condensing humidity equipment. To date, we have not noted any major change in these samples. We have noticed that some samples are changing color with a pink tinge. We have not observed this coloration in the field.

5 Conclusions

Of the three methods checked, comparing laboratory data with the requirements of ASTM standards does not permit the selection of the "best" membrane for use. Membranes represented by Samples 4, 11, 12, and 13, might be eliminated because of the loss of ~ 15% in elongation after heat aging, but this eliminates only one third of your candidates. This study shows that a core group of test requirements for all of the products serving the same use would enable the consumer to compare products offered for sale. Current ASTM standards do not permit this evaluation because they often use tests that cannot be compared, such as grab and strip tensile tests.

Rating these laboratory data and averaging these data provides an unbiased ranking, and a more rational approach to selection. Six of the PVC products rated highest of the 13 products tested.

Accelerated weather testing has little utility because it does not produce results fast enough to be practical (if useful results can be generated), does not take into account potential differences in failure mechanism (accelerated testing can influence one failure mechanism more than another), and the results achieved to date (such as the pinkish color seen in some samples) are not consistent with what we observed in the field.

The only rational system for selecting a roofing system is its past performance on the roof, in the same climate as the new project. If you must select a membrane system without the support of a history of excellent performance, an unbiased rating system may be of aid.

Table 1: Test data on PVC sheets and ASTM D4434 requirements

Sample number:	1	ASTM	2	3	4	5	ASTM	6	7	8	ASTM
ASTM type/grade:	II/1	II/1	III	III	III	III	III	IV	IV	IV	IV
Physical Test											
Caliper, mm	1.22	=/>1.14	1.30	1.24	1.17	1.22	=/>1.14	0.99	0.94	1.09	=/>0.91
(in.)	(0.048)	(=/>0.045	(0.051	(0.049	(0.046	(0.048	(=/>0.045	(0.039	(0.037	(0.043	=/>0.039)
Linear dimensional change											
% machine direction	0	=/<0.1	-0.15	-0.15	-0.2	-0.2	=/<0.5	-0.35	-0.15	-0.15	=/<0.5
% cross machine direction	-0.05	=/<0.1	-0.05	-0.05	-0.05	-0.05	=/<0.5	-0.05	-0.05	-0.1	=/<0.5
Water absorption, %	3.62	<+/-3	3.38	4.57	4.05	4.72	<+/-3	6.37	5.58	13.46	<+/-3
Cold Bend @ -40	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass
Tensile strength, grab method											
machine direction, kN	10791*	10343*	1.24	1.43	1.50	1.91	=/>0.89	1.66	1.99	1.61	=/>1.00
(lbf)	(1565)'	(1500)'	(279)	(321)	(337)	(429)	(=/> 200)	(372)	(447)	(363)	(=/> 275)
cross machine direction, N	10673*	10343*	1.30	1.37	1.31	1.56	=/>0.89	1.67	1.82	1.31	=/>1.00
(lbf)	(1548)'	(1500)'	(291)	(309)	(295)	(350)	(=/> 200)	(375)	(408)	(294)	(=/> 275)
Elongation at sheet breaking, %											
machine direction	310	=/>250	-	-	-	-	-	-	-	-	-
across machine direction	304	=/>220	-	-	-	-	-	-	-	-	-
Elongation at fiber breaking, %											
machine direction	-	-	34	43	33	38	=/>15	44	35	32	=/>25
across machine direction	-	-	44	39	39	34	=/>15	36	38	35	=/>25
Seam strength, % of tensile	>100	=/>75	>100	>100	>100	>100	=/>75	>100	>100	>100	=/>75
Effect of heat conditioning, 80° C (176°F) for six weeks											
Low temperature bend	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass
Tensile strength, % of original											
machine direction	112	=/>90	112	102	87	98	=/>90	93	106	99	=/>90
across machine direction	99	=/>90	99	99	98	107	=/>90	97	93	96	=/>90
Elongation, % of original or fiber breaking											
machine direction		=/>90	99	103	85	112	=/>90	91	109	109	=/>90
across machine direction		=/>90	100	99	103	116	=/>90	104	103	98	=/>90
Elongation, % of original or sheet breaking											
machine direction	108	=/>90	85	88	83	139	=/>90	105	98	113	=/>90
across machine direction	113	=/>90	97	100	94	123	=/>90	101	118	128	=/>90

Legend: * value is in kPa - ' value is (psi)

Table 2: Test data on "other" sheets and proposed TPO specification

Sample Number:	**9**	**10**	**11**	**12**	**13**	**ASTM**
Principal Polymer	ABC	PVC/EVA	TPO	EP	TPO	TPO
Physical Test						
Caliper, mm	1.17	0.86	1.09	1.09	1.14	=/>1.0
(in.)	(0.046)	(0.034)	(0.043)	(0.043)	(0.045)	(=/>0.039)
Linear dimensional change						
% machine direction	-0.1	-0.2	-0.49	-0.3	-0.55	=/<2
% cross machine direction	-0.05	0	-0.1	0	0.5	=/<2
Water absorption, %	14.4	16.14	4.09	6.47	10.37	<+/- 4
Cold bend @ -40°C (or F)	pass	pass	pass	pass	pass	pass
Tensile strength, grab method						
machine direction, N	2477	2252	1136	1205	1213	=/>1001
(lbf)	(557)	(506)	(225)	(271)	(273)	(=/>225)
cross machine direction, N	1432	2260	1559	903	999	=/>1001
(lbf)	(322)	(508)	(350)	(203)	(224)	(=/>225)
Elongation at fiber breaking, %						
machine direction	30	22	41	56	32	=/>15
across machine direction	37	29	40	62	38	=/>15
Seam strength, % of tensile	>100	>100	>100	>100	>100	=/>100
Effect of heat conditioning, 80° C (176°F) for six weeks						
Low temperature bend	pass	pass	pass	pass	pass	pass
Tensile strength, % of original						
machine direction	96	103	130	123	87.0	=/>100
across machine direction	107	105	98	153	86.0	=/>100
Elongation, % of fiber breaking						
machine direction	101	116	88	64	82	=/>100
across machine direction	96	121	90	80	109	=/>100
Elongation, % of sheet breaking						
machine direction	105	82	96	101	40	=/>100
across machine direction	109	105	94	85	82	=/>100

Table 3: Test data on all samples

Sample Number:	1	2	3	4	6	9	8	7	11	5	10	13	12
Principal Polymer:	PVC	PVC	PVC	PVC	PVC	ABC	PVC	PVC	TPO	PVC	PVC/EVA	TPO	EP
Physical Test													
Caliper, mm	1.22	1.30	1.24	1.17	0.99	1.17	1.09	0.94	1.09	1.22	0.86	1.14	1.09
(in.)	(0.048)	(0.051)	(0.049)	(0.046)	(0.039)	(0.046)	(0.043)	(0.037)	(0.043)	(0.048)	(0.034)	(0.045)	(0.043)
Compound thickness, optical													
Above reinforcing, mm	nt	0.554	0.512	0.410	0.436	0.476	0.485	0.312	0.457	0.439	0.317	0.418	0.347
(in.)	nt	(0.022)	(0.020)	(0.016)	(0.017)	(0.019)	(0.019)	(0.012)	(0.018)	(0.017)	(0.012)	(0.016)	(0.014)
Below reinforcing, mm	nt	0.425	0.381	0.490	0.250	0.406	0.481	0.199	0.427	0.359	0.302	0.471	0.383
(in.)	nt	(0.017)	(0.015)	(0.019)	(0.010)	(0.016)	(0.019)	(0.008)	(0.017)	(0.014)	(0.012)	(0.019)	(0.015)
Without reinforcing, mm	nt	1.249	1.128	1.010	0.860	1.170	1.086	0.717	1.096	1.094	0.850	1.170	0.789
(in.)	nt	(0.049)	(0.044)	(0.040)	(0.034)	(0.046)	(0.043)	(0.028)	(0.043)	(0.043)	(0.033)	(0.046)	(0.031)
Sheet thickness, optical mm	1.206	1.249	1.251	1.146	0.987	1.187	1.115	0.808	1.118	1.109	0.879	1.185	0.960
(in.)	(0.047)	(0.049)	(0.049)	(0.045)	(0.039)	(0.047)	(0.044)	(0.032)	(0.044)	(0.044)	(0.035)	(0.047)	(0.038)
Linear dimensional change													
% machine direction	0	-0.15	-0.15	-0.2	-0.35	-0.1	-0.15	-0.15	-0.49	-0.2	-0.2	-0.55	-0.3
% cross machine direction	-0.05	-0.05	-0.05	-0.05	-0.05	-0.05	-0.1	-0.05	-0.1	-0.05	0	+0.5	0
Water absorption, %	3.62	3.38	4.57	4.05	6.37	14.4	13.46	5.58	4.09	4.72	16.14	10.37	6.47
Analysis - THF extraction													
Mass, kg/m^2	1.52	1.55	1.46	1.40	1.21	1.54	1.45	1.06	1.01	1.55	1.07	1.09	0.98
(lb/100 ft^2)	(31.2)	(31.8)	(30)	(28.6)	(24.8)	(31.6)	(29.6)	(21.7)	(20.6)	(31.8)	(21.9)	(22.4)	(20.1)
Fabric, kg/m^2	0.06	0.07	0.09	0.09	0.12	nt	0.09	0.16	na	0.13	nt	nt	nt
(lb/100 ft^2)	(1.14)	(1.42)	(1.83)	(1.86)	(2.36)	nt	(1.93)	(3.19)	na	(2.62)	nt	nt	nt
Cold bend @ -40°C (or F)	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass
Tensile strength, grab method													
machine direction, N	nt	1241	1426	1500	1656	2477	1614	1986	1136	1919	2252	1213	1205
(lb)	nt	(279)	(321)	(337)	(372)	(557)	(363)	(447)	(255)	(429)	(506)	(273)	(271)
cross machine direction, N	nt	1295	1374	1313	1668	1432	1305	1816	1559	1557	2260	999	903
(lb)	nt	(291)	(309)	(295)	(375)	(322)	(294)	(408)	(350)	(350)	(508)	(224)	(203)
Elongation at sheet breaking, %													
machine direction	nt	136	134	141	127	84	110	119	152	90	80	321	149
across machine direction	nt	111	153	159	89	78	101	75	192	75	94	984	278

Legend: nt = not tested because solvent extraction was incomplete, or test was not appropriate.

Table 3 (continued): Test data on all samples

Sample Number:	1	2	3	4	6	9	8	7	11	5	10	13	12
Principal Polymer:	PVC	PVC	PVC	PVC	PVC	ABC	PVC	PVC	TPO	PVC	VC/EV	TPO	EP
Physical Test													
Elongation at fiber breaking, %													
machine direction		35	42	39	44	30	33	35	41	34	22	32	56
across machine direction		44	39	38	36	37	34	38	40	30	29	38	62
Tensile strength, strip method													
machine direction, kN/m	12	27	24	23	28	64	36	42	21	40	55	14	14
(lb/in. width)	(71)	(157)	(137)	(134)	(162)	(368)	(208)	(238)	(121)	(226)	(313)	(79)	(80)
cross machine direction, kN/m	12	22	24	26	36	30	28	28	24	30	49	8	12
(lb/in. width)	(70)	(126)	(138)	(146)	(204)	(173)	(156)	(159)	(138)	(172)	(281)	(43)	(69)
Elongation at break, %													
machine direction	310	31	30	27	29	28	32	29	24	34	26	19	26
cross machine direction	304	37	32	36	32	33	35	30	31	28	30	12	29
Die wicking													
machine direction, mm	0	0	130	0	0	85	0	18	0	57	45	0	85
(in.)	0	0	(5.12)	0	0	(3.35)	0	(0.71)	0	(2.24)	(1.77)	0	(3.35)
cross machine direction, mm	0	0	63	0	0	91	0	21	0	68	75	0	88
(in.)	0	0	(2.48)	0	0	(3.58)	0	(0.83)	0	(2.68)	(2.97)	0	(3.46)
Seam strength, % of tensile	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100	>100
Effect of heat conditioning, 80° C (176° F) for six weeks													
Low temperature bend	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass	pass
Tensile strength, % of original													
machine direction	112	112	102	87	93	96	99	106	130	98	103	87.0	123
across machine direction	99	99	99	98	97	107	96	93	98	107	105	86.0	153
Elongation, % of original or fiber breaking													
machine direction		99	103	85	91	101	109	109	88	112	116	82	64
across machine direction		100	99	103	104	96	98	103	90	116	121	109	80
Elongation, % of original or sheet breaking													
machine direction	108	85	88	83	105	105	113	98	96	139	82	40	101
across machine direction	113	97	100	94	101	109	128	118	94	123	105	82	85

Legend: nt = not tested because solvent extraction was incomplete

Table 4: Test data ranked for all samples

Sample Number:	1	2	3	4	6	9	8	7	11	5	10	13	12
Principal Polymer:	PVC	PVC	PVC	PVC	PVC	ABC	PVC	PVC	TPO	PVC	PVC/EVA	TPO	EP
Physical Test													
Caliper	94	100	90	76	96	90	84	73	84	94	67	88	84
Thickness above reinforcing	nt	100	74	79	92	86	88	56	83	79	57	76	63
Thickness below reinforcing	nt	87	100	51	78	83	98	41	87	73	62	96	78
Thickness between reinforcing	nt	100	81	69	90	94	87	57	88	88	68	94	63
Optical sheet thickness	96	100	92	79	100	95	89	65	89	89	70	95	77
Thermal stability, length	100	73	64	36	73	82	73	73	11	64	64	0	45
Thermal stability, width	90	90	90	90	90	90	80	90	80	90	100	0	100
Water absorption	78	79	75	61	72	11	17	65	75	71	0	36	60
Total mass	98	100	90	78	94	99	93	68	65	100	69	70	63
Fabric mass	36	45	58	74	57	nt	61	100	nt	82	nt	nt	nt
Cold bend @ -40	100	100	100	100	100	100	100	100	100	100	100	100	100
Grab tensile strength. md	nt	50	61	67	58	100	65	80	46	77	91	49	49
Grab tensile strength. xmd	nt	57	58	74	61	63	58	80	69	69	100	44	40
Elongation at sheet breaking, md	nt	42	44	40	42	26	34	37	47	28	25	100	46
Elongation at sheet breaking, xmd	nt	11	16	9	16	8	10	8	20	8	10	100	28
Elongation at fiber breaking, md	nt	63	70	79	75	54	59	63	73	61	39	57	100
Elongation at fiber breaking, xmd	nt	71	61	58	63	60	55	61	65	48	47	61	100
Tensile strength, strip method, md	19	43	36	44	37	100	56	65	33	61	85	21	22
Tensile strength, strip method, xmd	25	45	52	73	49	62	56	57	49	61	100	15	24
Elongation at maximum stress, md	100	10	9	9	10	9	10	9	8	11	8	6	8
Elongation at maximum stress, xmd	100	12	12	11	11	11	12	10	10	9	10	4	10
Die wicking, md	100	100	100	100	0	35	100	86	100	56	65	100	35
Die wicking, xmd	100	100	100	100	31	0	100	77	100	25	18	100	3
Seam strength	100	100	100	100	100	100	100	100	100	100	100	100	100
+Heat, low temperature bend	100	100	100	100	100	100	100	100	100	100	100	100	100
+Heat, tensile strength, md	60	60	57	77	93	87	97	80	0	93	90	57	23
+Heat tensile strength, xmd	98	98	96	94	98	87	92	87	96	87	91	74	0
+Heat elongation @ fiber breaking, md	nt	97	58	75	92	97	75	75	67	67	56	50	0
+Heat elongation @ fiber breaking, xmd	nt	100	86	81	95	81	90	86	52	24	0	57	5
+Heat elongation @ sheet break, md	87	75	72	92	80	92	78	97	93	35	70	0	98
+Heat elongation @ sheet break, xmd	55	89	79	96	100	68	0	36	79	18	82	36	46
Average, all ratings	82	74	70	70	69	69	68	67	66	63	61	60	52

Legend: nt = not tested (does not apply); md = machine direction; xmd = cross machine direction.

102 EFFECT OF APPLICATION TEMPERATURE ON ADHESION OF MODIFIED BITUMEN MEMBRANE ASSEMBLIES

K. Chernotowich

W.P. Hickman Systems, Inc., Solon, OH, USA

Abstract

Results are presented and discussed of a laboratory study which measures ply adhesion of polymer modified bitumen membranes assembled under a wide range of application conditions and temperatures. The evaluation includes both standard and styrene-ethylene-butylene-styrene (SEBS) modified asphalts. Additional factors examined include the role of mineral parting agents, the temperature of the individual sheets during membrane construction, as well as the effects of room temperature and dark oven heat aging at 70°C (158°F) of the completed assemblies. The findings indicate that the use of elevated bitumen application temperatures, previously thought to improve adhesion, has little or no effect on interply peel strengths. When considering the other deleterious consequences which may result from the elevated heating of asphalts, this approach may be considered more detrimental than beneficial to long term system performance.

Keywords: asphalt, heat aging, membrane assembly temperature, modified bitumen membranes, styrene-ethylene-butylene-styrene (SEBS) modified asphalt, T-peel adhesion

1 Introduction

Recently, there has been a movement to eliminate the use of the equiviscous temperature (EVT) concept for asphalt application as it pertains to the construction of polymer modified bitumen membrane systems. Simply stated, EVT represents the temperature at which asphalt achieves its proper viscosity for application. A couple

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examples indicating the move away from the use of EVT with modified bitumen systems include:

- The American Society for Testing and Materials (ASTM) recently modified the EVT definition in Standard Terminology Related to Roofing, Waterproofing, and Bituminous Materials (D 1079) to indicate that EVT may not be appropriate for use with polymer modified bitumen products.
- The National Roofing Contractors Association (NRCA) issued a position paper in late 1997, entitled "Blistering of SBS Modified Bitumen Membrane Systems," in which it is stated that EVT and EVT range are intended for built-up roofing, not polymer modified sheets. The NRCA believes that at EVT, there is not enough heat to achieve proper fusion to modified sheets, and consequently they encourage the use of higher temperatures.

This study was performed to examine these claims by evaluating incremental changes in application temperature, and considering other issues as they pertain to the elevated heating of asphalts.

2 Experimental details

All adhesion testing followed the guidelines of ASTM Test Method for Peel Resistance of Adhesives (T-Peel Test) (D 1876), using a crosshead speed (CHS) of 51 mm/min (2 in/min). All test samples were laboratory prepared to a size of 305 mm x 305 mm (12 in x 12 in), yielding ten 25.4 mm (1inch) by 305 mm specimens per assembly. Both machine and cross machine directions were examined. The sheet materials used were all polyester reinforced since glass fiber products fracture in the T-peel procedure. Specifics of the materials used are as follows.

- Base sheet which meets requirements of ASTM D 4601, Specification for Glass Fiber Base Sheet.
- 3 mm SBS modified bitumen smooth surface sheet typically used as a base ply in a polymer modified bitumen membrane system.
- 4 mm SBS modified bitumen mineral surfaced sheet typically used as the cap ply in a polymer modified bitumen membrane system.
- ASTM D 312, Type III asphalt, a primary industry selection for assembling polymer modified bituminous membrane systems.
- ASTM D 6152, SEBS modified asphalt, incorporated in the study to determine whether or not a polymer modified mopping asphalt would demonstrate any added benefits.

The initial test temperatures chosen were EVT, the extremes of EVT range [the standard ± 14°C (25°F) variable], and an additional 14°C (25°F) above the EVT range. Both asphalts in this study had an EVT of 218°C (425°F), thus yielding test temperatures of 204°C (400°F), 218°C (425°F), 232°C (450°F), and 246°C (475°F). The importance of these chosen temperatures is twofold.

First and foremost, the question of not using EVT during the assembly of a polymer modified bitumen membrane system has always been supported by the need for higher application temperatures to properly adhere or fuse these systems. Unfortunately, no specific guidelines have ever been proposed.

Secondly, and more importantly, the incorporated temperatures did focus on 1) EVT and EVT range, 2) what might reasonably be expected on the low side of temperature application, and 3) respect for flash point, a minimum 260°C (500°F) as specified in ASTM D 312. To maintain consistent 14°C (25°F) temperature intervals, this simply meant the addition of 190°C (375°F) as the lowest temperature, and 260°C (500°F) on the top end. With 190°C, it is not unreasonable to expect such a temperature in a cold climate application. As for 260°C, this was below the stated minimum flash points of 288°C (550°F) for the two asphalts of this study, staying the required minimum 14°C below flash point as recommended in the NRCA Roofing and Waterproofing Manual, Volume 1.

3 Results and Discussion

For the initial trial, the two systems chosen combined a standard 4 mm mineral surface MB cap sheet with either a standard asphalt base sheet or a 3 mm smooth surface MB sheet. Each system was then assembled with a standard Type III asphalt and a SEBS modified asphalt, yielding a total of four sample groups. Application temperatures used were 204, 218, 232, and 246°C. Testing was done after room temperature aging of 7 days, and 1, 3, and 6 months.

In Table 1, results for the standard base sheet/4 mm MB cap sheet configurations are presented. Each data point represents the average of 10 specimens, 5 each in machine and cross machine directions. Using confidence levels of 95% throughout this work, the respective data indicate no statistically significant gain is achieved with increasing application temperature, nor with the use of the SEBS modified mopping asphalt. What was clearly substantiated was that even at 204°C, there was enough available heat to thoroughly weld the mopping asphalt to the standard base sheet coating, creating a very clear cohesive failure within the base sheet's reinforcement/coating interface. This was the dominant failure mode, with the balance failing off the bottom of the MB cap sheet.

Microscopic examination of the tested specimens revealed that the failure mode with either mopping asphalt was tearing within the polyester reinforcing filaments. Increasing temperature did demonstrate a tendency to penetrate a bit deeper into the reinforcement; however, adhesion data revealed no significant change.

In Table 2, the T-peel adhesion data for the 3mm MB base/4 mm MB cap system present a different picture. Each data point represents the same cross section as in Table 1. Again, there essentially is no gain derived from using higher application temperatures with either asphalt type. However, there is marked improvement in adhesion with the SEBS asphalt. In most cases, these differences are statistically significant, particularly in average load values, the more critical measure. There are some data anomalies, but the basic trends are clear.

Table 1: Results of ASTM D 1876 T-Peel Adhesion Testing at 23°C (73°F) Specimens Aged at 23°C (73°F)

Assembly ID
Standard Base Sheet/4 mm MB Cap

		Load @ Peak, N (lbf)		Average Load Between Limits, N (lbf)	
	Asphalt Type ⟹	Type III	SEBS	Type III	SEBS
Bitumen Temperature	Days Aging				
204°C	7 Days	26.7 (6.0)	40.5 (9.1)	19.1 (4.3)	18.2 (4.1)
(400°F)	30 Days	25.6 (5.8)	38.9 (8.8)	18.2 (4.1)	19.8 (4.5)
	3 Months	24.7 (5.7)	38.7 (8.7)	12.5 (2.8)	19.8 (4.5)
	6 Months	22.0 (5.0)	34.2 (7.7)	10.9 (2.5)	19.1 (4.3)
	Average	24.8 (5.6)	38.1 (8.6)	15.2 (3.4)	19.2 (4.3)
218°C	7 Days	36.3 (8.2)	45.1 (10.2)	20.5 (4.6)	23.1 (5.2)
(425°F)	30 Days	23.8 (5.4)	42.3 (9.5)	17.6 (4.0)	22.2 (5.0)
	3 Months	31.6 (7.1)	41.6 (9.4)	16.7 (3.8)	19.6 (4.4)
	6 Months	22.0 (5.0)	39.8 (9.0)	12.2 (2.8)	21.4 (4.8)
	Average	28.4 (6.4)	42.2 (9.5)	16.7 (3.8)	21.6 (4.9)
232°C	7 Days	25.4 (5.7)	47.6 (10.7)	17.6 (4.0)	30.7 (6.9)
(450°F)	30 Days	33.8 (7.6)	38.7 (8.7)	21.4 (4.8)	21.4 (4.8)
	3 Months	28.9 (6.5)	40.5 (9.1)	18.5 (4.2)	20.2 (4.6)
	6 Months	26.5 (6.0)	37.8 (8.5)	15.6 (3.5)	18.9 (4.3)
	Average	28.6 (6.4)	41.2 (9.3)	18.2 (4.1)	22.8 (5.1)
246°C	7 Days	31.6 (7.1)	48.3 (10.9)	17.8 (4.0)	22.5 (5.1)
(475°F)	30 Days	20.7 (4.7)	43.8 (9.9)	14.2 (3.2)	21.1 (4.8)
	3 Months	31.6 (7.1)	45.8 (10.3)	17.1 (3.9)	20.5 (4.6)
	6 Months	24.5 (5.5)	40.0 (9.0)	12.9 (2.9)	18.9 (4.3)
	Average	27.1 (6.1)	44.4 (10.0)	10.0 (3.5)	20.7 (4.7)

Table 2: Results of ASTM D 1876 T-Peel Adhesion Testing at 23°C (73°F)
Specimens Aged at 23°C (73°F)

Assembly ID
3 mm MB Base/4 mm MB Cap

		Load @ Peak, N (lbf)		Average Load Between Limits, N (lbf)	
	Asphalt Type ⟹	Type III	SEBS	Type III	SEBS
Bitumen Temperature	Days Aging				
204°C	7 Days	45.1 (10.2)	87.8 (19.8)	36.3 (8.2)	67.6 (15.2)
(400°F)	30 Days	50.5 (11.4)	83.2 (18.7)	40.5 (9.1)	62.0 (14.0)
	3 Months	46.7 (10.5)	69.2 (15.6)	37.1 (8.4)	50.7 (11.4)
	6 Months	51.4 (11.6)	76.5 (17.2)	40.9 (9.2)	51.2 (11.5)
	Average	48.5 (10.9)	79.2 (17.8)	38.7 (8.7)	57.9 (13.0)
218°C	7 Days	48.5 (10.9)	117.9 (26.5)	35.6 (8.0)	78.7 (17.7)
(425°F)	30 Days	48.7 (11.0)	95.6 (21.5)	32.0 (7.2)	64.1 (14.4)
	3 Months	51.2 (11.5)	88.3 (19.9)	29.4 (6.6)	62.3 (14.0)
	6 Months	57.2 (12.9)	80.1 (18.0)	41.4 (9.3)	62.3 (14.0)
	Average	51.4 (11.6)	95.5 (21.5)	34.6 (7.8)	66.8 (15.0)
232°C	7 Days	33.6 (7.6)	45.8 (10.3)	22.0 (5.0)	37.6 (8.5)
(450°F)	30 Days	39.6 (8.9)	50.9 (11.5)	19.1 (4.3)	42.0 (9.5)
	3 Months	47.8 (10.8)	53.6 (12.1)	35.6 (8.0)	42.0 (9.5)
	6 Months	51.8 (11.7)	54.7 (12.3)	36.7 (8.3)	41.6 (9.4)
	Average	43.2 (9.7)	51.3 (11.5)	32.3 (7.3)	40.9 (9.2)
246°C	7 Days	44.5 (10.0)	78.1 (17.6)	31.6 (7.1)	51.8 (11.7)
(475°F)	30 Days	35.8 (8.1)	69.4 (15.6)	26.5 (6.0)	56.0 (12.6)
	3 Months	48.0 (10.8)	74.3 (16.7)	33.8 (7.6)	54.9 (12.4)
	6 Months	51.2 (11.5)	67.8 (15.3)	35.1 (7.9)	53.8 (12.1)
	Average	44.9 (10.1)	72.4 (16.3)	31.8 (7.1)	54.2 (12.2)

Microscopic evaluation of these samples was also very clear cut. Regardless of asphalt type or application temperature, all failure modes were of the adhesive type directly off one or the other MB face. Even at 246°C, there was not enough total available heat to truly fuse these sheets. Additionally, no cohesive failure occurred either within the mopping asphalt or either surface coating. All failures were adhesive and directly off one of the MB faces.

Based upon these results, additional series were planned to evaluate only the 3mm MB base/4 mm MB cap configuration. The limiting nature of the adhesion with the standard base sheet coating would restrict the development of useful information regarding application temperature. The initial results from the standard MB system also prompted an extension of the study as follows:

- The application temperature range was expanded from 190 to 260°C to examine a wider range of temperature extremes.
- Assemblies were prepared in four configurations.
    1) Standard sheets held at room temperature.
    2) Standard sheets preheated at 65°C (150°F) to simulate temperatures of membrane and roll goods during a sunny day.
    3) Sheets held at room temperature were brushed to remove excess mineral parting agent from each appropriate side.
    4) Sheets were preheated to 65°C, but brushed to remove excess mineral parting agent from each appropriate side.
- Added 35 days heat aging at 70°C per ASTM Standard Practice for Dark Oven Heat Exposure of Bituminous Materials (D 5869), to replace the extended room temperature aging.

Table 3 presents results for 7 days aging at room temperature. In this and the following table, each data point for the Type III membrane assemblies represent testing of twenty specimens, 5 each in machine and cross machine direction, and evaluated in duplicate trials. This was done to verify usage with Type III asphalt, the dominant adhesive used in the field. With respect to the SEBS asphalt assemblies, the data represent one complete series with 10 specimens, five from each direction.

Once again there is no consistent gain in adhesion with increasing application temperature, regardless of asphalt variable. Simply comparing values of the Type III assemblies at 190 and 260°C reveals statistical equivalence. Examination within any of the four categories involving the respective sheet variables reveals data with very little difference from top-to-bottom. However, as before, there are substantial and significant increases in adhesion when using the SEBS modified mopping asphalt. These results also compare very well with those shown in Table 2. When employing a two standard deviation range to yield 95% confidence levels, most of the data in Tables 2 and 3 were found to be statistically equivalent.

When the results from dark oven heat aging shown in Table 4 are compared with the data of Table 3, a significant decrease in adhesion is found with the Type III asphalt, again equally across all sheet variables and application temperatures. These data represent an approximate decrease of 50%, and application temperature provided no relevant positive impact. Regarding the data for the SEBS modified asphalt assemblies; these too show no impact from increasing application temperature. However, they differ in that adhesion did not substantially decrease across all samples. In many cases, the data are essentially equivalent to those from room temperature aging (Table 3). When comparing the average load data for the two asphalts after heat aging, the SEBS specimens clearly demonstrate statistically improved adhesion.

Visual examination of the tested specimens from Tables 3 and 4 found the dominant failure mode to be adhesive directly off one of the MB faces. There was a fair amount of cohesive failure; these occurring exclusively with the SEBS modified asphalt. Interestingly, the level of cohesive failures increased substantially after the 35 days of heat exposure. Once again these occurred exclusively with the SEBS asphalt assemblies.

Finally, as an overview from the assembly phase, one very key aspect was the significant effect on the mopping asphalts that increasing temperatures clearly creates. As application temperature increased, asphalt viscosity decreased, and a self-leveling aspect was clearly evident. This results in a naturally thinner asphalt layer. More importantly, however, is that the increasing temperature also brings a faster cooling rate within the poured asphalt. Essentially, elevated application temperatures are doubly non-productive in that they induce lighter interply quantities that also have a tendency to cool faster, particularly at the surface where we are hoping to achieve better bonding with the MB sheets. This finding generates a host of additional points or questions that go beyond the scope of this paper.

4 Conclusions

Results of this evaluation, encompassing two asphalt types, a variety of sheet variables, and both room temperature and heat aging conditions, clearly do not support the premise that simply raising application temperature will induce better fusion with polymer modified bitumen sheets. Other indicated results are as follows.

- Removal of the parting agent did not appear to help bonding of these products.
- The use of SEBS modified asphalt demonstrated statistically improved bonding of the polymer modified sheets.
- Heat aging of the standard Type III asphalt assembled specimens demonstrated a marked decline in levels of adhesion.
- Heat aging of the SEBS modified asphalt assembled specimens exhibited equivalent adhesion to those cured at room temperature.

To achieve better service life and durability, we agree that a primary goal should be to create a more thoroughly bonded system. With increasing adhesion comes a stronger, unified system more resistant to failure. However, simply elevating application temperatures beyond asphalt's EVT and EVT range are not the answer. In fact, elevated temperatures can bring about several negative aspects that include the following points.

Table 3: Results of ASTM D 1876 T-Peel Adhesion Testing at 23°C (73°F)
Specimens Aged 7 Days at 23°C (73°F)

Assembly ID
3 mm MB Base/4 mm MB Cap

	Load @ Peak, N (lbf)		Average Load Between Limits, N (lbf)	
Asphalt Type ⟹	Type III	SEBS	Type III	SEBS
Standard Sheets @ Ambient Temp.				
Bitumen Temp. @				
190°C (375°F)	37.8 (8.5)	111.3 (25.1)	28.3 (6.4)	89.4 (20.1)
204°C (400°F)	40.0 (9.0)	110.6 (24.9)	22.3 (5.0)	87.2 (19.6)
218°C (425°F)	35.4 (8.0)	110.7 (24.9)	26.0 (5.9)	72.9 (16.4)
232°C (450°F)	31.7 (7.1)	64.3 (14.5)	37.9 (8.5)	50.1 (11.3)
246°C (475°F)	36.4 (8.2)	76.7 (17.3)	28.9 (6.5)	51.1 (11.5)
260°C (500°F)	43.2 (9.7)	70.9 (16.0)	33.7 (7.6)	45.8 (10.3)
Standard Sheets @65°C (150°F)				
Bitumen Temp. @				
190°C (375°F)	48.9 (11.0)	102.8 (23.2)	37.5 (8.5)	84.6 (19.1)
204°C (400°F)	38.3 (8.6)	85.6 (19.3)	26.6 (6.0)	65.2 (14.7)
218°C (425°F)	45.0 (10.1)	95.7 (21.5)	31.1 (7.0)	65.5 (20.0)
232°C (450°F)	42.3 (9.6)	87.0 (19.6)	31.8 (7.2)	62.7 (14.1)
246°C (475°F)	43.9 (9.9)	71.7 (16.1)	33.7 (7.6)	48.0 (10.8)
260°C (500°F)	39.9 (9.0)	77.4 (17.4)	28.1 (6.4)	52.2 (11.8)
Brushed Sheets @ Ambient Temp.				
Bitumen Temp. @				
190°C (375°F)	47.5 (10.7)	109.1 (24.6)	31.2 (7.0)	70.2 (15.8)
204°C (400°F)	35.4 (8.0)	114.8 (25.8)	22.8 (5.1)	83.4 (18.8)
218°C (425°F)	33.9 (7.6)	107.4 (24.2)	34.3 (7.7)	72.9 (16.4)
232°C (450°F)	45.2 (9.4)	97.6 (22.0)	32.2 (7.3)	60.4 (13.6)
246°C (475°F)	40.4 (9.1)	79.9 (18.0)	29.0 (6.5)	58.0 (13.1)
260°C (500°F)	39.9 (9.0)	69.5 (15.7)	35.9 (8.1)	52.2 (11.7)
Brushed Sheets @65°C (150°F)				
Bitumen Temp. @				
190°C (375°F)	46.3 (10.4)	95.2 (21.4)	35.9 (8.1)	72.7 (16.4)
204°C (400°F)	40.4 (9.1)	94.1 (21.2)	29.1 (6.6)	76.5 (17.2)
218°C (425°F)	45.8 (10.3)	93.1 (21.0)	39.1 (8.8)	59.1 (13.3)
232°C (450°F)	44.1 (9.9)	93.5 (21.0)	30.6 (6.9)	70.8 (15.9)
246°C (475°F)	41.6 (9.4)	78.5 (17.7)	27.9 (6.4)	49.5 (11.1)
260°C (500°F)	49.3 (11.1)	79.7 (18.0)	36.6 (8.3)	55.6 (12.5)

**Table 4: Results of ASTM D 1876 T-Peel Adhesion Testing at 23°C (73°F)
Specimens Aged 35 Days at 70°C (158°F)**

Assembly ID
3 mm MB Base/4 mm MB Cap

	Load @ Peak, N (lbf)		Average Load Between Limits, N (lbf)	
Asphalt Type ⇒	Type III	SEBS	Type III	SEBS
Standard Sheets @ Ambient Temp. Bitumen Temp. @				
190°C (375°F)	19.0 (4.3)	78.3 (17.6)	14.2 (3.2)	57.7 (13.0)
204°C (400°F)	19.9 (4.5)	62.7 (14.1)	14.0 (3.2)	26.8 (6.1)
218°C (425°F)	21.1 (4.7)	82.8 (18.7)	14.0 (3.2)	62.2 (14.0)
232°C (450°F)	19.3 (4.4)	74.7 (16.8)	13.3 (3.0)	53.5 (12.1)
246°C (475°F)	25.1 (5.6)	84.4 (19.0)	17.8 (4.0)	63.2 (14.2)
260°C (500°F)	25.5 (5.4)	85.9 (19.3)	18.0 (4.0)	62.7 (14.1)
Standard Sheets @65°C (150°F) Bitumen Temp. @				
190°C (375°F)	16.1 (3.7)	95.3 (21.4)	9.9 (2.2)	82.2 (18.5)
204°C (400°F)	19.8 (4.5)	87.3 (19.6)	14.7 (3.3)	57.9 (13.0)
218°C (425°F)	23.4 (5.2)	63.7 (14.4)	16.2 (3.6)	45.7 (10.3)
232°C (450°F)	18.3 (4.1)	87.8 (19.7)	12.9 (2.9)	62.7 (14.1)
246°C (475°F)	19.3 (4.3)	84.1 (18.9)	14.7 (3.3)	56.3 (12.7)
260°C (500°F)	23.5 (5.3)	93.2 (21.0)	16.1 (3.6)	74.5 (16.8)
Brushed Sheets @ Ambient Temp. Bitumen Temp. @				
190°C (375°F)	21.9 (4.9)	81.2 (18.3)	17.0 (3.8)	54.7 (12.3)
204°C (400°F)	21.2 (4.7)	92.1 (20.7)	16.9 (3.8)	68.2 (15.4)
218°C (425°F)	22.6 (5.1)	77.9 (17.5)	16.8 (3.8)	59.0 (13.3)
232°C (450°F)	20.7 (4.7)	86.1 (19.4)	15.4 (3.4)	64.6 (14.5)
246°C (475°F)	23.9 (5.4)	80.6 (18.1)	17.2 (3.8)	62.0 (14.0)
260°C (500°F)	31.5 (7.1)	79.6 (17.9)	19.2 (4.3)	57.7 (13.0)
Brushed Sheets @65°C (150°F) Bitumen Temp. @				
190°C (375°F)	19.1 (4.3)	76.1 (17.2)	13.2 (3.0)	50.3 (11.3)
204°C (400°F)	18.4 (4.1)	102.0 (23.0)	14.5 (3.2)	70.2 (15.8)
218°C (425°F)	22.9 (5.2)	79.3 (17.8)	16.9 (3.8)	61.1 (13.7)
232°C (450°F)	20.4 (4.6)	87.5 (19.7)	14.9 (3.4)	66.8 (15.1)
246°C (475°F)	21.6 (4.9)	85.8 (19.3)	16.0 (3.6)	56.7 (12.8)
260°C (500°F)	30.9 (6.9)	82.8 (18.6)	19.7 (4.4)	63.1 (14.2)

- Increasing the likelihood of additional oxidation or inducing softening point fallback in the asphalt.
- Enhancing the possibilities of generating carcinogenic pollutants that NIOSH and OSHA claim occurs with excessive heating.
- Elevating fume levels could very well increase the opportunity for blister formation by entrapping bitumen volatiles.
- Higher temperatures that decrease asphalt viscosity yield decreased interply quantities that negatively impact adhesion capabilities and total waterproofing.
- If fallback occurs, an even greater, more permanent reduction in viscosity and other key asphalt properties can result in a detrimental contribution to a system's overall (long-term) performance and service life.

5 Recommendations

Our findings indicate that asphalt EVT and EVT range are still very relevant parameters when assembling a polymer modified bitumen membrane system. The increase of application temperature does not bring about improved bonding of the sheet products. Alternately, it does provide the opportunity for negatively impacting the work environment and compromising the durability and service life of the membrane system.

A positive result of this work is the indication that SEBS modified asphalt provides enhanced adhesion in these systems, and does so within the guidelines of standard EVT and EVT range. Clearly, if increased system adhesion is determined to be a key property of a polymer modified membrane system, SEBS modified asphalts should be given serious consideration.

6 References

Annual Book of ASTM Standards, (1998) Volumes 04.04 and 15.06, American Society for Testing and Materials

National Roofing Contractors Association, (1997) *Blistering of SBS Modified Bitumen Membrane Systems*

National Roofing Contractors Association, (1996) *NRCA Roofing and Waterproofing Manual, Volume 1, Fourth Edition*

103 SERVICE LIFE PREDICTION OF FLAT ROOFS WITH POLYMER MODIFIED BITUMINOUS WATERPROOFING MEMBRANES

S. Croce,[1] E. De Angelis,[1] A. Tiso[2]

[1]Politecnico di Milano, Italy; [2]UCLA - CONICIT, Venezuela

Abstract

The paper proposes a model for the prediction of flat roof service life based on the evaluation of the durability of the bituminous waterproofing membrane through accelerated thermal aging tests and its characterisation with Dynamic Mechanical Analysis (carried out with Rheometrics RDAII apparatus). Two different qualities of polymer modified (APP) bituminous waterproofing membranes were tested and these results have been used to measure performance level and to predict service life.
DMA tests on waterproofing membranes give information about the material's mechanical behavior that can be related to its molecular structure and can be further related to the basic decay process and kinetic reactions relationship. Using the results of accelerated thermal aging test of waterproofing membrane, Service Life can be calculated for each flat roof type and climate context.

Keywords: flat roof, waterproofing membranes, service life, thermal decay, durability of building materials, polymers modified bituminous membranes.

1 Durability and service life

Durability is generally assumed to be the ability of a building product to withstand weathering factors and to limit decay processes it can be subject to. This generic concept can be translated in something less general and more measurable that can be linked to the service life of the element or system the product is part of it.

If one wants to evaluate the service life of a critical system such as a flat roof, one can start with identifying the most probable failure modes. Then, one can set

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apart those failure modes that are brought about by design or construction or maintenance errors.

If one is able to identify the basic weathering mechanisms of its components that give rise to "non-pathological" failure modes of the system, then one can define a component's durability in terms of its weathering resistance and one can evaluate the service life of the system as function of this resistance and, as well, the weathering action specific to context the component is subjected to.

The variations of a specific characteristic Z with time (t) can be modeled as linear function of a steady weathering action level C (as in the case of friction weathering) or as an exponential function of it (this is the case of certain mechanical characteristics of waterproofing membrane with thermal aging). In these cases one write:

$$\text{linear model: } \Delta Z = \left(1/\delta_C\right) \cdot \Delta t \qquad \textbf{(1)}$$

$$\text{exponential model: } \Delta Z/Z_0 = \left(1/\delta_C\right) \cdot \Delta t \qquad \textbf{(1 bis)}$$

Where: Z_0 is the initial characteristic value of the tested component; Z is the actual value of the characteristic value at general time t; $1/C$ is the decay rate of the characteristic Z in a generic environmental stress condition "C" (weathering factor level) and δ_C its durability characteristic.

2 Service life prediction

Briefly, to predict the service life of a complex building system it is necessary: (1) to identify the failure modes; (2) to understand their qualitative and quantitative link with each decay process involved; (3) to model these elementary decay processes and find a relation with them and weathering factor levels.

The first and second point can be, in some way, resolved but the solution to the third point is not easily found through mere statistics. Natural decay process, in fact, can be longer than the life of the products studied on the market, hindering any feedback from field data in the prediction of decay processes. Indeed, innovation in the industry can produce better products (with lower natural decay rates) before one is able to evaluate the service lives of the first generation products.

Accelerated aging tests can be performed, but they have a fundamental limitation, that is, accelerated aging conditions cannot be easily related to in-service conditions. To overcome this problem, time transformation functions have to be identified, in order to relate accelerated weathering condition to in-service weathering conditions and, correspondingly, accelerated time to normal time. A time transformation function will be able to relate different weathering conditions to different time periods for the same amount of weathering. Infact, whatever model one chooses (linear, with eq. 1 or exponential, with eq. 1 bis), one can write the following equivalence:

$$\Delta t = \frac{\delta_C}{\delta_{C'}} \cdot \Delta t' = K_{C-C'} \cdot \Delta t' \qquad \textbf{(2)}$$

Where: $K_{C\text{-}C'}$ is the time transformation function from the C to the C' weathering factor level.

Then we can generalise previous equation and write the following expression for the evaluation of service life, on the basis of a time transformation functions:

$$SL = K \cdot t_{AST} \tag{3}$$

Where: SL is the predicted Service Life; t_{AST} is the time-to-failure, measured in a standard accelerated weathering conditions (defined for the specific failure mode); K, is the time transformation function that relates accelerated decay rate to the decay rate produced by average local conditions (for a non-protected product exposed to local micro climate).

3 Waterproofing membrane characteristics and flat roofing service life

The case of flat roofs and polymer-modified bituminous waterproofing membranes is straight forward, as the principal failure mode of the first is the failure of the waterproofing membrane (water infiltration through it) and it is normally connected, in non-pathological cases, to one weathering factor: bituminous compound embrittlement by thermal aging, i.e. an embrittled membrane is no longer capable withstanding normal mechanical stresses.

The nature of polymer-bituminous compound, in fact, makes the membrane very sensitive to thermal exposure: chemical reactions of the compound induced by thermal energy reduce the mobility between molecular chains and induce a growth in internal cohesion forces. This change in its micro-structure generates changes in the mechanical behavior of the compound (i.e. glass transition temperature increases) and consequently, alters the performance of the membrane. If one assume that the ageing decay rate will follow an exponential Arrhenius relationship:

$$1/\delta_C = A \cdot \exp(-B/T) \tag{4}$$

Where: A (h^{-1}) is a constant referred as the material failure mechanism; T (°K) is the weathering factor level (thermal stress condition); $B=E/k$ (°K) is a fixed value related to the composition of the material and the specific thermal induced decay reactions, where E is the thermal energy of activation of the reaction and k is Boltzmann's constant.

Then, the time transformation function from standard test conditions (T_{test}) to average in-service conditions (T) can be written as:

$$K = \frac{A \cdot \exp(-B/T_{test})}{A \cdot \exp(-B/T)} = \frac{\exp(-B/T_{test})}{\exp(-B/T)} \tag{5}$$

4 Modeling of thermal stress

Waterproofing thermal exposure is related to local weather conditions and to roof layer sequence, as they can provide more or less thermal protection to the membrane, because of their construction sequence. Thermal decay rate has been expressed as a function of temperature (4). As temperature is constant only in thermal aging tests, and in-service conditions are characterised by variable temperature cycles due to the weather conditions, we have to find a significant temperature value.

Temperature can be generally assumed as a periodical variable and expressed as a function of some parameters. The following equation is proposed:

$$T(t) = \overline{T}_y + 0{,}5 \cdot \Delta T_a \cdot (1 - \cos(\pi \cdot t/4380)) + 0{,}5 \cdot \Delta T_{d(t)} \cdot (1 - \cos(\pi \cdot t/12)) \qquad \textbf{(6)}$$

Where: t is the time in hours; $\overline{T}_y$, is the mean yearly air temperature; ΔT_a, is the difference between the maximum (summer) and minimum (winter) mean daily temperature, in an annual cycle; $\Delta T_{d(t)}$ is the daily temperature difference, that changes in accordance with the seasons. The amplitudes of the two periodical components can be expressed as:

$$\Delta T_{d(t)} = 0.5 \cdot (\Delta T_{dS} + \Delta T_{dw}) - 0.5 \cdot (\Delta T_{dS} - \Delta T_{dw}) \cdot \cos\left(\pi \cdot t/_{4380}\right) \qquad \textbf{(7)}$$

$$\Delta T_a = \Delta T_T - \Delta T_d \qquad \textbf{(8)}$$

$$\Delta T_T = \Delta Ts_{max} - \Delta Tw_{min} \qquad \textbf{(9)}$$

Where: ΔT_{dS}, is the difference between the maximum and minimum hourly temperature in a standard summer day; ΔT_{dW}, is the difference between the maximum and minimum hourly temperature in a standard winter day; ΔTs_{max}, is the maximum value of average hourly temperature in summer; Tw_{min}, is the minimum value of average hourly temperature in winter; ΔT_d, is the mean difference between ΔT_{dS} and ΔT_{dW}.

Based on the above relations, a frequency distribution series can be calculated to given temperature intervals, given by the percentage of the hours in a yearly cycle.

To be able to generalise the application to any thermal condition a membrane might be subjected to, the temperature distribution is related to $\Delta T_d / \Delta T_T$; in this regard, the results of the percentages of hours in a year are shown in Table 1.

Table 1: Frequency distribution in percentages of hourly temperatures that can be registered from Tw_{min} to Ts_{max} in a annual cycle, assuming a sinusoidal daily and yearly variation of the temperature values in function of ΔT_d and ΔT_T.

Temperature Intervals			$\Delta T_d/\Delta T_T$										
From		*To*	*0*	*0.1*	*0.2*	*0.3*	*0.4*	*0.5*	*0.6*	*0.7*	*0.8*	*0.9*	*1*
$Tw_{min}+0.9\Delta T_T$	→	Ts_{max}	25	15	8	7	6	6	6	5	7	8	21
$Tw_{min}+0.8\Delta T_T$	→	$Tw_{min}+0.9\Delta T_T$	0	10	11	6	5	6	6	6	7	14	8
$Tw_{min}+0.7\Delta T_T$	→	$Tw_{min}+0.9\Delta T_T$	17	12	14	13	9	7	5	9	10	10	8
$Tw_{min}+0.6\Delta T_T$	→	$Tw_{min}+0.7\Delta T_T$	0	6	8	9	10	8	9	8	11	9	8
$Tw_{min}+0.5\Delta T_T$	→	$Tw_{min}+0.6\Delta T_T$	8	5	7	11	14	12	10	11	10	8	4
$Tw_{min}+0.4\Delta T_T$	→	$Tw_{min}+0.5\Delta T_T$	8	11	9	6	8	14	14	14	9	8	4
$Tw_{min}+0.3\Delta T_T$	→	$Tw_{min}+0.4\Delta T_T$	0	0	3	10	13	11	20	14	12	8	8
$Tw_{min}+0.2\Delta T_T$	→	$Tw_{min}+0.3\Delta T_T$	17	17	14	8	10	16	12	14	12	10	8
$Tw_{min}+0.1\Delta T_T$	→	$Tw_{min}+0.2\Delta T_T$	0	0	6	12	15	11	9	10	13	12	8
Tw_{min}	→	$Tw_{min}+0.1\Delta T_T$	25	25	20	16	11	9	8	9	9	14	21

After this, we can use equation (5) to evaluate K, assuming that the total effect of the thermal decay rate as the addition of each effect in the decay rate given by each step of thermal level. So, we have an expression that can be used along with the values given by Table 1, with each percentage and temperature value:

$$K_T = \frac{\exp\left(-B/T_{test}\right)}{\sum_{i=1}^{n} \frac{\%_i}{100} \exp\left(-B/T_i\right)} \tag{10}$$

Where: $\%_i$ is the percentage of hours in a annual cycle at determinate temperature values; T_i, is the nominal temperature value related to every interval, and it can be assumed with the value of the maximum limit superior of the temperature interval.

5 Dynamic mechanical analysis (DMA)

As mentioned before, waterproofing membrane is the most critical component of a flat roof systems and thermal decay is the principal bituminous membrane alteration process. Any bituminous blend or composite is characterised by a temperature range in which the material is considered performing. The uppest limit of this range is the hottest temperature without excessive softening of the compound (it can be evaluated with the ring & ball test as it is usually done for bituminous blend) and it is usually evaluated through the ring & ball test. The lower limit - the coldest temperature at which the membrane can maintain good flexible behavior - is usually evaluated through the cold bending membrane standard test, an empirical test which shows the coldest temperature at which it can bend in a predetermined cylinder without cracking.

As these tests evaluate actual membrane behavior, the assessment of its durability is usually carried out measuring cold bending temperature evolution after different accelerated thermal aging periods tests at constant temperature exposition (70°C). Smaller changes of this characteristic between aged and unaged samples means longer waterproofing membrane durability. Nevertheless, this evaluation does not give information about the micro-structural conformation of the material and the fundamental failure mechanisms cannot be modeled to forecast decay rate.

Research is dealing with new testing methodologies, as for instance the Dynamic Mechanical Analysis (DMA) applied to waterproofing membrane thermal aging: this test, in fact, can provide information about the mechanical behavior of the compound over a large range of temperatures and more precise data about the variations of the thermal-mechanical characteristics against the fundamental changes in the material's micro-structure, that can be used more effectively to evaluate waterproofing durability and roof systems service life.

DMA tests consist of torso-deformation cycles of load on material samples to evaluate the characteristics of viscous-elastic-plastic response of the submitted material at a determined frequency. It provides the values of "storage shear module" G' and "dynamic shear module" G". The first is related to the potential energy that is stored as a mechanical response of the material, while the second one is related to the energy that is dissipated by heat. From the ratio of these material's modules (G"/G') the material's damping is given, this characteristic is related to the material's microstructure and the relative mobility between molecular chains in the material.

Carrying out DMA over a wide range of temperature values, with the help of a thermal cell and temperature control systems (available in Rheometrics RDII apparatus used in standard test for plastic materials) one can obtain a series of damping-temperature curves for various waterproofing membranes. These damping-temperature values provide interesting and useful information about the thermal-mechanical behavior of waterproofing membrane; changes in damping values after aging can be easily related to decay reactions and weathering process of the material.

6 Experimental decay rates of waterproofing membranes

A series of experimental program tests were carried out based on accelerated thermal aging of polymer modified bitumen waterproofing membranes: the aim was the experimental evaluation of the changes of mechanical characteristics of the membrane generated by thermal exposure.

Two different qualities of polymer modified bituminous waterproofing membrane have been tested. The first type of waterproofing membrane (n.1) had a minimal amount of polymers and poor reinforcement, whereas the second one (n.2) was richer in polymers and better reinforced.

Table 2: Values of the characteristics of waterproofing membrane over different lengths of time of thermal ageing, and the respective decay rates and correltion parameters with respect to their tendency to follow an exponential model

Membrane 1	**Days of ageing at 70°C**				**decay**	**parameter**
Tests	**0 d.**	**28 d.**	**90 d.**	**180 d.**	**rate**	**R^2**
Cold Bending (°C)	0	10	15	25	0,0084	0,754
Softening Point Blend (°C)	143,0	143,5	146,0	146,0	0,0001	0,7869
Max. Tens. Strength Long. (N)	471	612	759	648	0,0015	0,3698
Max. Elongation Long. (%)	28	8	5	5	-0,0079	0,5951
Damping at 0°C (adim.)	0,34	0,29	0,23	0,20	-0,0029	0,9328
Damping at 20°C (adim.)	0,38	0,35	0,28	0,24	-0,0026	0,9701
Membrane 2	**Days of ageing at 70°C**				**decay**	**parameter**
Tests	**0 d.**	**28 d.**	**90 d.**	**180 d.**	**rate**	**R^2**
Cold Bending (°C)	-15	-10	-5	5	0,0081	0,9038
Softening Point Blend (°C)	152,0	152,0	152,0	153,0	0,0000	0,7777
Max. Tens. Strength Long. (N)	824	861	868	812	-0,0001	0,1269
Max. Elongation Long. (%)	48	41	41	40	-0,0008	0,5128
Damping at 0°C (adim.)	0,28	0,22	0,20	0,20	-0,0017	0,6643
Damping at 20°C (adim.)	0,30	0,25	0,22	0,22	-0,0016	0,7416

Standard and DMA membrane tests were performed on submitted unaged and aged samples of both materials, after 28, 90 and 180 days of accelerated thermal aging at 70 °C (UNI-8202/27). The results of these tests for both materials are shown in Table 2. Moreover the decay rate and its correlation parameter (R^2) are included, both calculated according to an exponential tendency curve of the respective characteristic vs. time.

In accordance with the results of standard tests, membrane n. 2 can be considered more durable than membrane n. 1, as the first shows lower changes in the values of its Cold Bending Temperature against thermal aging.

First results confirm that an exponential model can be proposed for thermal decay process of polymer modified bituminous membranes. In absence of any time-transformation function, the values of δ_C should only be obtained empirically from field experiences. In this sense, results taken from standard accelerated thermal aging tests at constant temperature of 70 °C can be used to calculate the decay rate of the respective characteristics of the waterproofing membranes studied and the parameters used into the model considering the parameters when they are in service.

7 Roof service life prediction

Equation n.3 can be used to estimate the service life of a roof waterproofing system. We have applied this method to two different roofs in different localities: Milan (45°N) and Maracaibo (10° N). Input data and results are shown in Table 3, for the following roofs:

1. unprotected membrane in a 20 cm thick concrete roof deck, without insulation.
2. unprotected membrane in a 20 cm thick concrete roof deck, 5 cm layer insul.
3. protected waterproofing in a 20 cm thick concrete roof deck, 5 cm layer insul.
4. inverted roof with waterproofing membrane located under 5 cm insulation over a 20 cm thick concrete roof deck.

The distribution of temperatures for the different exposure of the membrane analyzed in the examples are showed in Table 4 (Milan) and Table 5 (Maracaibo) based on the distribution given by table 1. Taking as a reference the experimental results of tests on membrane 1 (which showed a higher decay degree and lost its main characteristics over 180 days of thermal aging testing at constant temperature of 70°C) we calculated *K* using Eq. 9 for the different types of roof, assuming B= 8,000 K. With these values of *K*, we calculated the expected service life of the different roofs, which are shown in Table 6.

8 Conclusions

The results obtained confirm the importance of controlling thermal exposure and the durability performance of the membrane for a longer roof service life; especially for site conditions characterised by high temperature peaks and hig solar irradiation.

The model presented can give only an indication of the expected service life of a roof, through analysis of thermal decay rates. Nevertheless, these results can be useful used in building design and planning when service life expectancy data is not available. Further investigation is required in order to obtain a more accurate prediction; but this may not be useful for standard applications.

Table 3: Application of two locations of different latitude in calculating the relationship $\Delta T_d/\Delta T_T$ to the membrane temperature values as a function of the roof technologies

Site	*Year Period*	*Temp. Values*	*Roof Type 1*	*Roof Type 2*	*Roof Type 3*	*Roof Type 4*
Milan **Lat.: 45° N**	Coldest month, winter (January)	Tw_{max} (°C)	10	10	8	15
		Tw_{min} (°C)	0	-3	0	15
		ΔT_{dw}(°C)	10	13	8	0
	Hottest month, summer (June)	Ts_{max} (°C)	52	60	50	23
		Ts_{min} (°C)	22	19	23	22
		ΔT_{ds} (°C)	30	41	27	1
		$\Delta T_d = (\Delta T_{dw}+\Delta T_{ds})/2 =$	20	27	17	1
		$\Delta T_T = Ts_{max} - Tw_{min} =$	52	63	50	8
		$\Delta T_d/\Delta T_T =$	***0.38***	***0.43***	***0.34***	***0.13***
Maracaibo **Lat.: 10° N**	Coldest month, winter (January)	Tw_{max} (°C)	55	65	53	25
		Tw_{min} (°C)	25	23	25	24
		ΔT_{dw}(°C)	30	42	28	1
	Hottest month, summer (June)	Ts_{max} (°C)	60	72	58	25
		Ts_{min} (°C)	28	26	28	24
		ΔT_{ds} (°C)	32	46	30	1
		$\Delta T_d = (\Delta T_{dw}+\Delta T_{ds})/2 =$	31	44	29	1
		$\Delta T_T = Ts_{max} - Tw_{min} =$	35	49	32	1
		$\Delta T_d/\Delta T_T =$	***0.89***	***0.88***	***0.90***	***1.00***

Table 4: Assumed membrane temperature frequency distribution of the different types of roofs in the city of Milan (Lat. 45° N), according to the application of the values given by tables 2 and 3

Roof Type 1		*Roof Type 2*		*Roof Type 3*		*Roof Type 4*	
$\Delta T_d/\Delta T_T = 0.38 \approx 0.4$		$\Delta T_d/\Delta T_T = 0.43 \approx 0.4$		$\Delta T_d/\Delta T_T = 0.34 \approx 0.3$		$\Delta T_d/\Delta T_T = 0.13 \approx 0.1$	
T_i (°C)	$\%_i$	T_i (°C)	$\%_i$	T_i (°C)	$\%_i$	T_i (°C)	$\%_i$
$Ts_{max} = 52$	6	$Ts_{max} = 60$	6	$Ts_{max} = 50$	7	$Ts_{max} = 23$	15
46.8	5	53.7	5	45.0	6	22.2	10
41.6	9	47.4	9	40.0	13	21.4	12
36.4	10	41.1	10	35.0	9	20.6	6
31.2	14	34.8	14	30.0	11	19.8	5
$T_{med} = 26.0$	8	$T_{med} = 28.5$	8	$T_{med} = 25.0$	6	$T_{med} = 19.0$	11
20.8	13	22.2	13	20.0	10	18.2	0
15.6	10	15.9	10	15.0	8	17.4	17
10.4	15	9.6	15	10.0	12	16.6	0
5.2	11	3.3	11	5.0	16	15.8	25
$Tw_{min} = 0$		$Tw_{min} = -3$		$Tw_{min} = 0$		$Tw_{min} = 15$	

Table 5: Assumed membrane temperature frequency distribution of the different types of roofs in the city of Maracaibo (Lat. 10° N), according to the application of the values given by Tables 2 and 3

Roof Type 1		*Roof Type 2*		*Roof Type 3*		*Roof Type 4*	
$\Delta T_d/\Delta T_T = 0.89 \approx 0.9$		$\Delta T_d/\Delta T_T = 0.88 \approx 0.9$		$\Delta T_d/\Delta T_T = 0.90$		$\Delta T_d/\Delta T_T = 1.0$	
T_i (°C)	$\%_i$	T_i (°C)	$\%_i$	T_i (°C)	$\%_i$	T_i (°C)	$\%_i$
$Ts_{max} = 60$	8	$Ts_{max} = 72$	8	$Ts_{max} = 57$	8	$Ts_{max} = 25$	21
56.5	14	67.1	14	53.8	14	24.9	8
53.0	10	62.2	10	50.6	10	24.8	8
49.5	9	57.3	9	47.4	9	24.7	8
46.0	8	52.4	8	44.2	8	24.6	4
$T_{med} = 42.5$	8	$T_{med} = 47.5$	8	$T_{med} = 41.0$	8	$T_{med} = 24.5$	4
39.0	8	42.6	8	37.8	8	24.4	8
35.5	10	37.7	10	34.6	10	24.3	8
32.0	12	32.8	12	31.4	12	24.2	8
28.5	14	27.9	14	28.2	14	24.1	21
$Tw_{min} = 25$		$Tw_{min} = 23$		$Tw_{min} = 25$		$Tw_{min} = 24$	

Table 6: Predicted service life for both waterproofing membranes, calculated as a function of the different conditions of assumed thermal exposure

Thermal Context	*Milan: Lat. 45° N* *T_{min}(air): 0°C.*				*Maracaibo: Lat. 10° N* *T_{min}(air): 20°C.*			
Reference Material Characteristic	*Membrane Damping at 0°C*				*Membrane Damping at 20°C*			
Technological Choice	*Roof 1*	*Roof 2*	*Roof 3*	*Roof 4*	*Roof 1*	*Roof 2*	*Roof 3*	*Roof 4*
*Calculated parameter, K_T**	*16.73*	*10.74*	*17.38*	*55.13*	*5.12*	*2.69*	*6.06*	*35.96*
S.L. Membrane n. 1 (years)	8.25	5.30	8.57	27.19	2.82	1.48	3.33	19.78
S.L. Membrane n. 2 (years)**	14.07	9.04	14.62	46.38	4.57	2.41	5.42	32.14

* *Calculated by equation 5 with temperature and percentage values given by Tables 4 and 5.*

** *Calculated by relations of the reference characteristic decay rates shown in table 2.*

9 References

Ang Aik Leng. "Prediction of the Remaining Service Life of Roofs: An Exponential Mathematical Model". Architectural Science Review, Vol. 34, September, 1991.

Cash C. "Estimated Mean Temperatures of Horizontal Surfaces for Predicting the Durability of Thermally Sensitive Materials (Arrhenius Relationship)". Proceeding of Dealing with Defects in Buildings. Varenna, Italy, September, 1994.

CIB & RILEM "Thermal Analysis Testing of Roofing Membrane Materials". Final report of the Thermal Analysis Task Group RILEM 120-MRS/CIB W83. March, 1995.

Gaddy G., Rossiter W., Eby R. "The Applicability of Modern Analytical Techniques to Detection of Changes in Roofing Membrane Materials brought about by Heat Aging". Proceeding of International Waterproofing Association. Amsterdam, 1995.

Nelson W. "Accelerated Testing: Statistical Models, Test Plans, and Data Analyses". John Wiley & Sons, 1990.

Oba K., Partl M.N. "Durability of Polymer-Modified Bituminous Roofing Membranes". Proceeding of Durability of Building Material & Components. Vol. II. Stockholm, Sweden, 19-23 May 1996.

Ozkan. "Temperatures Attained in Naturally Weathered Waterproofing Membranes Used for Roofing". Architecture and Building Science-King Saud University, Riyadh. Architectural Science Review, Vol. 34, March, 1991.

Tiso A., Di Landro L., Pagano M. "Stima della durabilitá di membrane impermeabilizzanti in bitume-polimero". I Convegno Nazionale sulla Scienza e Tecnologia dei Materiali. Lerici, Italia, Aprile, 1997.

Tomiita T. "Solar UV, Wetness and Thermal Degradation Maps in Japan". Proceeding of the Fifth International Conference of Durability of Building Materials. Brighton, UK, 7-9 November 1990.

104 EXTENDING THE DURABILITY OF WINDOW WASHER RUNWAYS

J. Henshell and P.L. Buccelato

Henshell and Buccellato, Consulting Architects, Red Bank, NJ, USA

Abstract

Glass curtain walls on many high rise buildings are washed from swinging scaffolds suspended from window washer carriages that circumnavigate the perimeter of the roof travelling on a concrete runway. No matter how well the runway is constructed and maintained, its durability is shortened by the need to remove it in part or in whole to repair or replace the roofing under it.

Most roofing systems are designed to perform for approximately 20 years – about one-third the life of the average building. Thus, removing the runway to reroof, because of its size and weight, becomes a logistical and fiscal disaster.

This paper proposes a solution to extend the life of the runway to approximate that of the building. It describes a case study using precast concrete panels assembled to form a runway that can be removed and reinstalled for membrane maintenance and replacement without the radical surgery of demolishing and disposing of cast-in-place paving. The panels are prefabricated complete with insulation, steel curbs and tie-downs. When assembled in configurations to conform to the parapet profile they form a runway that can support the moving load of the window washer rig, resist torque stresses and accommodate thermally induced movement.

Keywords: runway, precast concrete panels, window washing, roofing.

1 Introduction

Forty years ago, Lever House, the New York City glass box, ushered in the era of high rise buildings clad with metal and glass curtain walls. Soon similar buildings became a part of the landacape in every major city. Accelerated by the rapid improvement of HVAC systems, these buildings had few, if any operable sash which presented a problem for cleaning the glass and maintaining caulking.

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The earliest systems that evolved were simple swinging scaffolds supported either from movable beams resting on the roof or from cables connected to rings secured to the roof deck. Buildings with many setbacks used permanently mounted davits. Although these systems are still used on many buildings, they have been largely superseded by more sophisticated systems to reduce labor and increase safety.

One of the most common of these is an electric powered self-propelled carriage with a scaffold suspended from its davits. By raising or lowering the scaffold and moving the carriage, the entire facade is accessible for cleaning and maintenance. The carriage can be mounted on rails or tracks or travel on a concrete runway. It is commonly housed on the roof in "garages" for protection and maintenance.

2 Concrete window washer runways

Self-propelled carriages travelling on concrete runways are more predominant than track-mounted units. Runways are constructed of reinforced concrete slab, approximately 3 m (10 ft) wide, that follow the line of the parapet around the building perimeter. The outboard side terminates within a few centimeters of the interior face of the parapet. The inboard side is provided with a 100 mm (4 in.) curb to prevent the carriage from straying off the runway. (Fig. 1) Rings or prefabricated flush mounted tie-downs are installed on or near the curb to help counterbalance the scaffold. The curb has scuppers or drains at regular intervals.

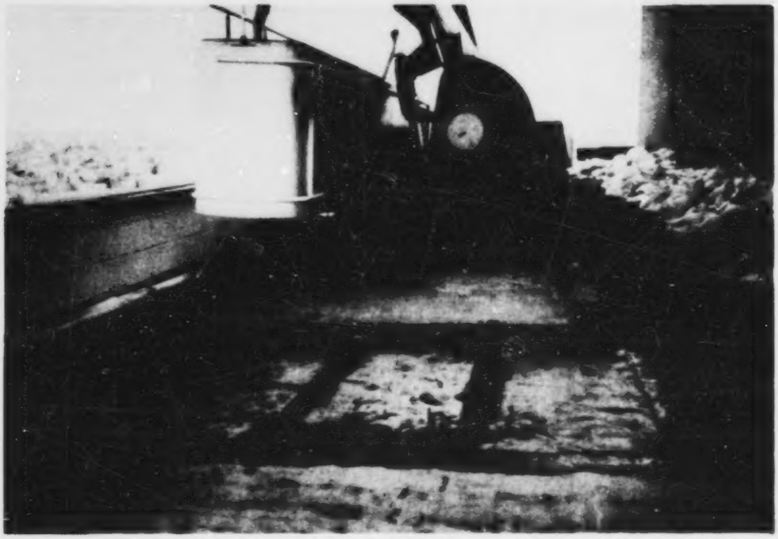

Fig 1: Window washer carriage on partly demolished runway

Most slabs are cast in 3 m (10 ft) lengths with 25 mm (1 in.) joints. These are filled with a preformed expansion joint filler and caulked. The caulking may be extruded in the summer where it is often damaged by the carriage's wheels. The joints are intended to absorb thermally induced movement and are thus dynamic. Since the slab, itself is dynamic, its movement can stress the membrane and abrade or tear the base flashing. More serious problems develop when the slab is cast around tie-down anchors that are anchored to the deck structure through the roofing system. Slab movement often ruptures the inaccessible flashing at the penetration.

Until a few decades ago the membrane was constructed as a bitumen and felt built-up roof (BUR) installed over insulation or directly on the concrete deck if insulation was not required. A variations of this system is a PMR configuration in which the membrane is installed on the roof deck, covered with loose-laid extruded polystyrene insulation and ballasted with aggregate or pavers. Contemporary membranes run the gamut of liquid applied membranes, BURs with polymer modified bitumen plys and loose-laid thermoplastic and thermosetting single ply sheets. In the absence of a PMR system, window washer runway slabs are cast over preformed asphalt/felt protection boards laid dry or mopped to the membrane.

3 Durability problems

Cast-in-place concrete paving proved to be a satisfactory surface for the window washer carriage providing it was properly reinforced and of good quality concrete (which was not always the case). From a roof maintenance and replacement standpoint, it was another story.

Most high rise buildings are designed to last from 50 to 75 years and some for as long as a century. On the other hand, it is generally agreed that roofs are considered to perform successfully if they can be maintained leak-free for 20 years. Thus, an average roof on an average high rise building will require replacement at least once and more than likely twice before the building has outlived its useful life. When this occurs, the concrete runway becomes a white elephant–it is too big and too expensive to remove and incapable of being made watertight for a reasonable time when the roof below it leaks.

If roof replacement is not required, roof maintenance creates another problem. The slab inhibits access to the flashing for repairs because it is cast against, or in close proximity to the parapet base flashing. And repairs are required more frequently because the slab movement often damages metal flashing end joints or abrades composition flashing as discussed above.

In an effort to avoid the financial and logistical headaches of removing some or all of the runway to effectuate repairs, attempts have been made to seal the runway and thus stop water infiltration into the concealed penetrations and flashing below it. Every one we observed had failed prematurely. Although the field of the runway can be "waterproofed", the expansion joints, the joint between the runway and the parapet flashing and the joint between the runway curb and roofing remain dynamic and well beyond the elastic capabilities of liquid coatings. Elastomeric sheet materials are equally ineffective because they are vulnerable to damage from the carriage's wheels when they rub against the parapet or twist over the expansion joints. Tie-downs that are cast into the runway are particularly difficult to flash to the membrane and almost impossible to waterproof at the slab surface.

The authors have investigated concrete window washer runways on nine high rise buildings in New York, New Jersey and Louisiana, all in connection with reported leaks in the underlying roofing and flashing. In all but two buildings, the owners rejected runway removal as too costly and we devised stop-gap methods short term methods to control the leaks. On one, because the leaks were pervasive throughout the roof, all components were removed to the deck and interlocking precast pavers were used to cover the entire roof.

There was no indecision about removing the runway on the other building– Salomon Smith Barney's headquarters in New York. In 1994 a roof leak had shorted out the electrical switchgear located under the roof causing a reputed $40,000,000 loss on the trading floor in three hours. This, coupled with the fact that the runway was in poor condition, constituted an overwhelming argument for removing it in preparation for reroofing the building. In agreeing to do so, the owners posed one question: We want to make every effort to assure that the roof will not leak again, but what will we do if it does?" Not wishing to replicate the white elephant on the roof, we opted to solve the problem of membrane maintenance under the runway by designing one that could be removed and replaced at a small fraction of the cost of the cast-in-place runway it would replace. This paper discusses its design and construction.

4 Case study

The building, constructed in 1986-7, is a 9 story reinforced concrete structure located on the Hudson River in lower Manhattan. The main roof is a 58 x 113 m (192 x 372 ft) rectangle containing a main penthouse, four stair bulkheads and dunnage mounted cooling towers.

The concrete roof deck is low sloped to internal drains and enclosed with a 1372 mm (54 in.) high parapet. It was roofed with a modified bitumen liquid applied membrane that was covered with extruded polystyrene insulation boards and pavers. The concrete window washer runway was 2.74 m (9 ft) wide, 150 mm (6 in.) thick with a 100 mm (4 in.) wide x 100 mm (4" in.) high curb. It was cast on the insulation boards over the membrane. The surface of the runway was badly scaling and the edges of the expansion joints were ravelling. (Fig. 2)

4.1 Design

The concept of using large precast concrete slabs on a roof was based on our prior experience designing 1.06 x 2.12 m (3.5 x 7 ft) precast concrete panels on the fourth level of a parking deck over a hospital operating suite. This was the subject of a paper presented at an ASTM Symposium in 1991 and reported in ASTM STP 1084.

For the parking deck project, we researched small 609 and 915 m square (24 and 36 in.) pavers and found them to be unsuitable for vehicular traffic. Problems of resisting braking loads and jacking loads had been only partly solved by Chaplin,

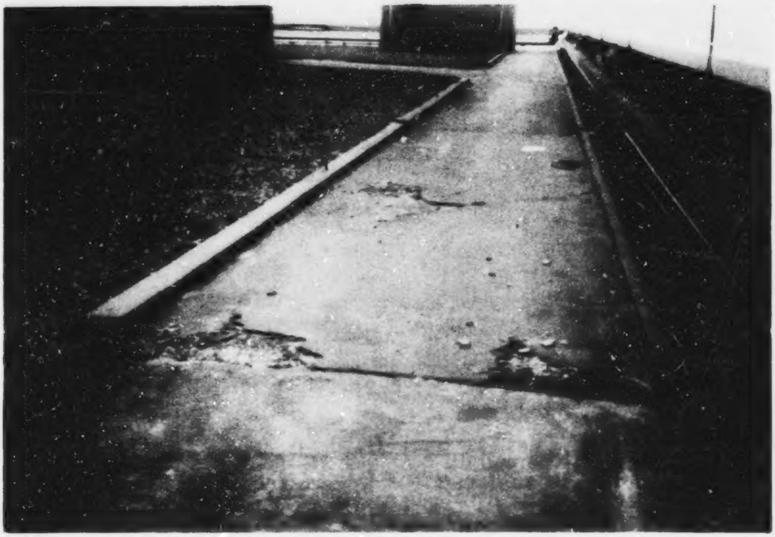

Fig. 2: Deteriorated runway

Eggle and Buckingham in England in 1978. Our research for the parking deck lead us to decide on the 1.06 x 2.12 m (3.5 x 7 ft) panels as being the most suitable size and weight for handling and function. One advantage of the larger panels was that they were sufficiently heavy to resist rocking when subjected to eccentric point loads. This and other lessons we learned from the parking deck project enabled us to fine tune the design of the panels for the window washer runway .

For the runway we designed panels that were 1.2 x 2.4 m (4 x 8 ft) with a nominal panel thickness of 127 mm (5 in.) nominal. This panel size was heavy enough to perform satisfactorily, but not so heavy that it could not be transported with simple hand and mechanical equipment such as fork lifts and slings. The unit weight was also consistent with the weight of the cast-in-place runway it replaced. Designed concrete compressive strength was 28 Mpa (5500 psi) at 28 days. Maximum absorption was 5% by weight and 10% by volume. (Fig. 3)

Sheets of extruded polystyrene insulation were recessed into the bottom of the panel and inset 50 mm (2 in.) from the edges. They provided the following advantages:

- reduced weight
- isolate the concrete from the membrane
- permitted drainage under and around the panels
- enhanced panel shimming

Precast concrete panels for a window washer runway differ in one significant respect from those used for a parking deck; they must be designed to resist the uplift forces on the inboard side produced by the eccentric loading of the scaffold. These can be quite sizeable and well in excess of the counterbalancing dead load of a single panel. To accommodate this load we reinforced the panel longitudinally to resist bending and interlocked adjacent panels to spread the uplift forces. For practical reasons the connecting plates on the inboard edge of the panel were used for interlocking rather than attempting to transfer the load to along the longitudinal edges of the panel by tongue and groove or splined joints. The plates were less expensive than complicated form inserts, the edges were able to move independently and differentially and the panels could be installed with greater flexibility.

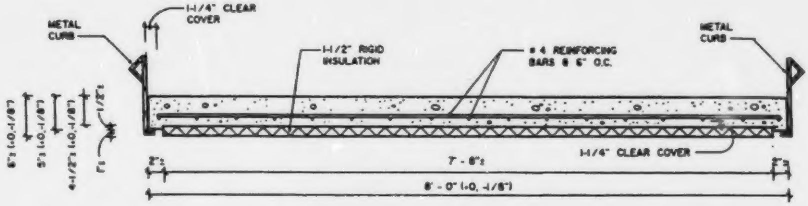

Fig. 3: Longitudinal section through precast runway panel

The runway was laid out with the long dimension normal to the parapet. (Fig. 4) It was important that the panel length be in one piece for the full width of the runway in order to act as a beam under negative loading. A spacer was provided between adjacent panels to provide adjustments and accommodate tolerances in the cast product and accommodate panel expansion and contraction.

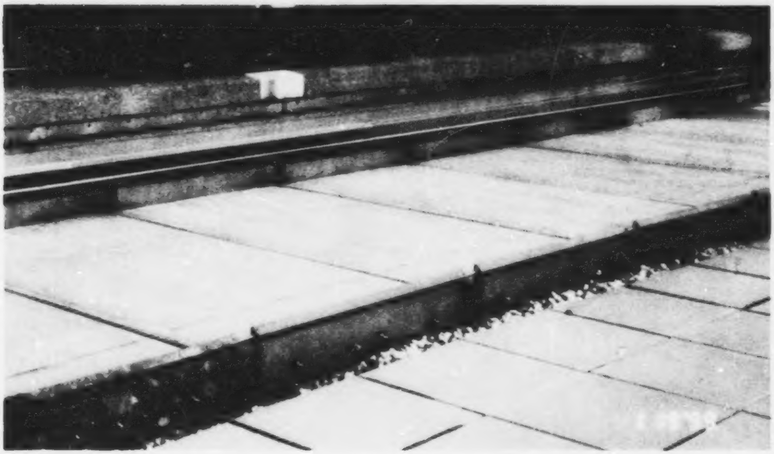

Fig 4: Arrangement of panels

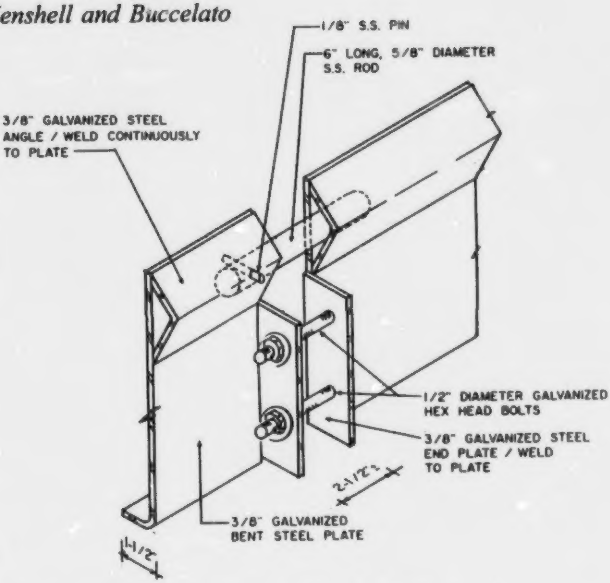
1/8" S.S. PIN
6" LONG, 5/8" DIAMETER S.S. ROD
3/8" GALVANIZED STEEL ANGLE / WELD CONTINUOUSLY TO PLATE
1/2" DIAMETER GALVANIZED HEX HEAD BOLTS
3/8" GALVANIZED STEEL END PLATE / WELD TO PLATE
2-1/2"
3/8" GALVANIZED BENT STEEL PLATE
1-1/2"

Fig. 5: Detail of curb end and interlock between panels

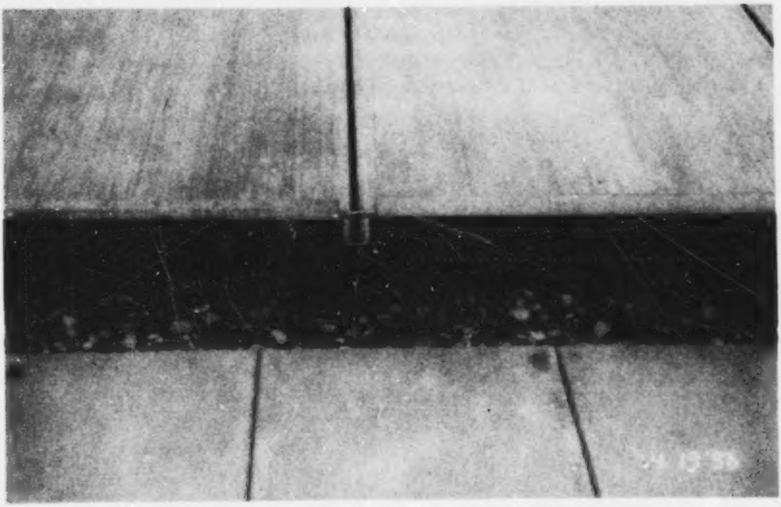

Fig. 6: Photo of curb interlock

One cast-in-place unit was provided in each run to absorb the differences in length from installation tolerances after the panels were in place. Joints were filled with backer rods and caulked to reduce lippage. Cast-in-place triangular units were provided at reentrant angles to accommodate the carriage's turning radius.

Bent steel plates were provided on the short sides of the panel. These served to maintain panel alignment and uniform joint widths during expansion and contraction of the panel groups, transfer stress between adjacent panels to increase the mass during the application of uplift force at the tie-down rings and act as a curb in lieu of the traditional cast-in-place concrete curb. The plates were provided with holes for connecting the tie-down cables during operation of the scaffold. (Figs. 5 and 6)

Other features were adapted from the panels designed for the parking garage project. Top surfaces were slightly crowned, but in only one direction to minimize transitions that the smaller carriage wheels would have to negotiate. The top surface was broomed and the edges were tooled convex.

4.2 Installation

Panels were precast in a casting plant under controlled conditions and air-cured. They were then loaded on a truck and shipped to the jobsite for weekend erection.

Two panels at a time were lifted to the top of the 9 story building by a crane and then distributed by dollies the prepared membrane surfaces. The membrane was covered by a single layer of asphalt protection board with the polyethylene side turned down. Pieces of protection board were used to shim the panels. (Fig. 7)

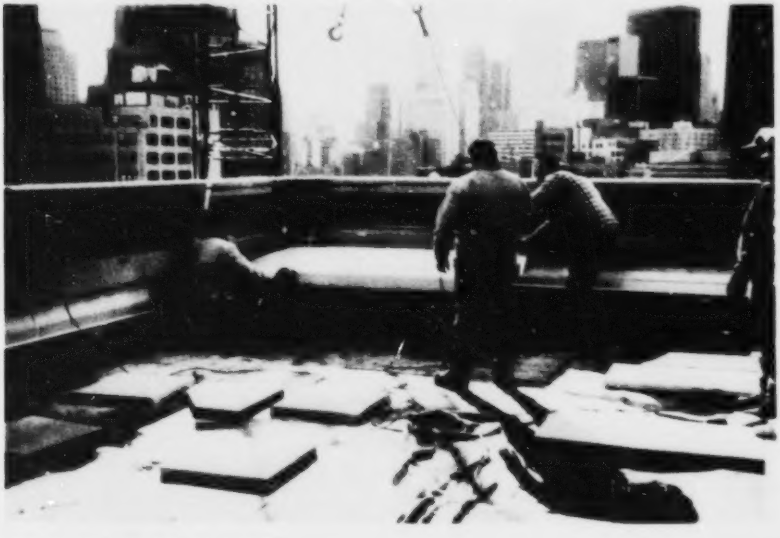

Fig. 7: Installation of a panel

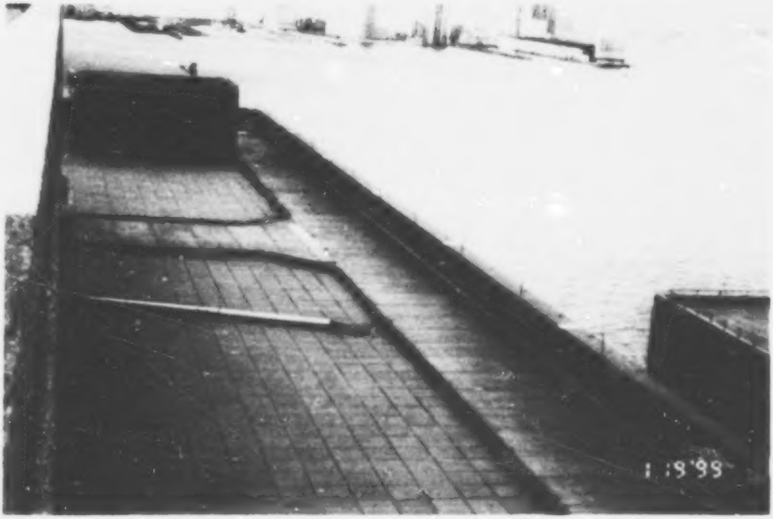

Fig. 8: The completed runway

5 Conclusions

Precast concrete panels for window washer runways provide the following advantages:

- they create runways independent of the roofing and flashing systems
- they can be removed and reinstalled as required for roof maintenance and replacement
- they incorporate tie-down anchors that do not penetrate the roofing system
- if panels are damaged or deteriorate individual units can be easily replaced

Based on lessons learned from previous attempts to repair membranes and flashings under window washer runways, we believe that constructing runways from precast units in lieu of using cast-in-place concrete can improve the durability of the roofing system and increase the life of the runway to nearly equal that of the building.

6 References

Chaplin, R. G. "Development of a Precast Concrete Paving Flag System for Inverted Roof Parking Decks" (1981) Proceedings, Second International Symposium on Roofs and Roofing, Vol 2, pp.75-84.

Eggle, J. A. and Buckingham, A. B. (1981) "Experience with an Inverted Car Deck Roof" Proceedings, Second International Symposium on Roofs and Roofing, Vol 1, pp. 345-354.

Henshell, J. and Buccellato, P. L, (1990) "Precast Concrete Pavers for Parking Decks," Building Deck Weatherproofing, American Society for Testing and Materials STP 1084, L. E. Gish, Ed. ASTM; Philadelphia, PA, pp. 83-94

105 EXPERIMENTAL PROGRAM FOR THE EVALUATION OF COIL COATED STEEL SHEET SERVICE LIFE

F. Re Cecconi and F. Ravetta

Polytechnic of Milan, Italy

Abstract

It is recognized that the best way to evaluate the actual service life of materials and components is to analyze statistical data from existing buildings; the observation of the behavior of existing structures and buildings requires a considerable amount of both economical and human resources, that in the majority of cases are not easily found. This paper is intended as a contribution in the field of the studies on the durability of building materials: it shows the main interim results from an ongoing research project at the Polytechnic of Milan. The project is in association with an Italian industry leader in coil coated steel sheet manufacturing. Basically, the industry in question employs the coil coated steel sheet to produce multilayer curtain components for roofs or walls. The results presented in this paper were obtained in one of the first test application of a new methodology elaborated at the DISET - Polytechnic of Milan, to evaluate the service life of building components. In particular, the results presented in the paper are derived from surveys on existing buildings built during the 1985 to 1996 period, and from three years of tests on atmospheric environmental exposure performed both in Italy and in Holland. The study analyzes several industrial buildings located in Italy, both in the northern and southern parts of the country. These results, obtained from a test application in controlled conditions, represent a useful tool both to tune the test methodology and gain additional knowledge in the field of the durability of coil coated steel sheet.

Keywords: Coil coated steel sheet, exposure test, service life, survey

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1 Introduction

At the DISET - Polytechnic of Milan more than ten years ago research on quality of building components and materials started and, at the present time, have produced a methodology to evaluate building components *technological quality*, i.e. the ability of a building component to satisfy the required performances at the beginning of its life, a methodology to compute *on site quality*, i.e. the ability of the component to satisfy all the requirements stated by the persons working on site, and a methodology to evaluate *reliability* of building components, as presented in Rejna 1996.

Work is now focused on a methodology to evaluate building components service life. This methodology, though already presented at an international symposium, Daniotti 1998, is still being revised and results coming from ongoing experimental programs are useful to refine it. In particular the methodology is divided into two main steps: first of all the evaluation of building products sensitivity and then the computation of building components service life starting from the sensitivity of the products. The first step of the methodology is strongly based upon bibliographical information on building materials behaviour over time. A lack of information about building products behaviour over time, particularly on coil coated steel sheet behaviour, made us start our own test program.

This paper presents some of the results achieved in the first phase of the research on building components service life, particularly on coil coated steel sheet.

2 Methodology description

The goal chosen for the research has been the evaluation of the durability of a set of seventeen building components for exterior wall without considering, in the first time, the connection among components. The first step of the methodology is the one that comes out with the evaluation of building products sensitivity that is essential in order to evaluate building components service life according to our methodology. The step that describes how to compute the sensitivity of building product is one of the most tested of the whole methodology and has been already presented in an Italian congress.

The evaluation of building components sensitivity, if they are imagined to be exposed directly to aging agents, can be made following this scheme:

1. definition of the aging agents relevant for the building product under evaluation;
2. collection of bibliographical information;
3. computation of building product sensitivity;

2.1 Definition of the aging agents

One of the first results of the research has been the definition of a set of aging agents to which the evaluation of building components behavior over time can be referred. It stands to reason that durability of building components changes according to the different environment (i.e. aging agents) in which they are used. Since it is possible that different building products are influenced in their behavior by different agents, the set defined has been used as a base point in order to create a subset of aging agents relevant for the specific product. Table 1 shows the set of agents, the ones relevant for coil coated steel sheet are highlighted using the italic. The set of agents here quoted has been defined joining indication of some Italian and foreign standards (English and American).

Table 1: Main set of agents.

Class of aging agents	Agent
Chemical agents	CO_x
	NO_x
	SO_x
	Sulfuric acid
	Carbonic acid
	Salts
	Cleaning products
Weathering agents	*Moisture*
	Rain
	Snow
	Ice
	Hail
	Water vapor
	Wind
	Air pollutants
	High temperature
	Low temperature
	Temperature cycle
	Solar radiation
	Thermal radiation
External artificial agents	Electromagnetic radiation
	Discontinuous mechanical stresses
Artificial agent due to use	Cleaning water
	Continuous mechanical stresses
Biological agents	Animals
	Vegetable

2.2 Collection of bibliographical information

Collecting data on behavior over time of building has been an essential step in the development of the research. The results of this phase have been stored in file-cards in order to have easy to read documents. Each file-card describes one action, i.e. the consequence of one or more agents on the building product. The cards are divided in fields describing possible effects of the action and chemical or physical phenomena that lead to the effects. Unfortunately there aren't information or studies on every possible action on building products and on the consequences of these actions, especially for innovative products. As a consequence of this lack of information we felt necessary to gain further data through specific test programs and surveys on existing buildings. In order to obtain more data on coil coated steel sheet we joined an Italian industry leader in coil coated steel sheet manufacturing, La Magona d'Italia S.p.A., which kindly allowed us to publish results obtained from:

- atmospheric environmental exposure tests;
- survey on existing buildings built since 1985 with their coil coated steel sheet

3 Atmospheric environmental exposure test results

Among the results obtained by the cooperation between DISET and La Magona d'Italia we can count those concerning atmospheric environmental exposure of coil coated steel sheet samples with different kind of coating. Samples with three different kind of coatings and painted with two different colors, light the first and dark the second, have been exposed for three years in two different places. The first exposure place is in Italy, in the nearby of the factory, the second is in Holland by the North Sea.

The three coatings differ for their resistance to the atmospheric agents, because of a different composition of the coating layer. In particular the manufacturer assures his products to have a color difference from the new ones less than 8 Celiab units (ECCA T3) and a chalking value (ASTM D659) higher than 6 after 10 years for the product called *system 3000/90*, after 15 years for the product called *system 5000/90* and after 20 years for the product called *system 10000/90*. The two different colors have been chosen in order to highlight eventual discoloration phenomena that are generally more emphasized on dark pigments. The following parameters were recorded:

- *D Gloss*: the gloss of a sample is measured by the amount of light reflected from a specimen in the specular direction. The shift value is reported as a percent value of the original gloss;
- *Scratch or edge*: rust penetration either through a cut on the coating layer or from the edge where the steel sheet is cut is measured. The value is reported as a percent value considering a penetration of 10 mm as 100%;
- *6mm hole*: rust penetration from the edge of a 6 mm diameter hole is measured. The value is reported as a percent value considering a penetration of 10 mm as 100%;
- *Bend 0-T*: this test comprises bending the sample through 180° over a period of 1-2 seconds around various radii. The value reported is the percentage of defective surface;
- *Erichsen 6mm*: coil coated steel sheet are subjected to ball punch deformation. The value reported is the percentage of defective surface;
- *Dust retention*: The value is reported as a percentage considering the value 10 measured according to ASTM D3274 as 100% and the value 0 as 0%;
- *Chalking*: The value is reported as a percentage considering the value 10 measured according to ASTM D659 as 100% and the value 0 as 0%;
- *Blistering*: The value reported is the percentage of defective surface evaluated according to ASTM D714

Unfortunately it wasn't possible to report the same aging parameters both for the Italian samples and for the Dutch ones because the exposure tests in Italy began before the exposure tests in Holland and followed test methodologies based upon Italian standards, while in the Dutch tests the European Coil Coated Association standards were followed.

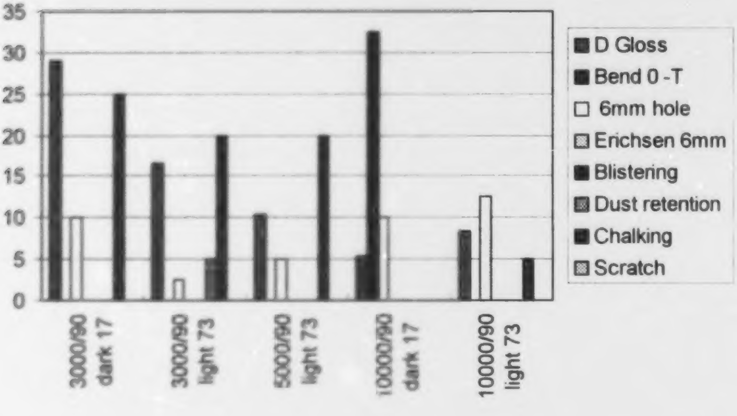

Figure 1: Results after 36 months of exposure in Italy

Examining the results after 36 months of exposure in Italy (figure 1) we can see that the principal defects are a generalized loss of the initial gloss, some chalking and rust attack on the edge of the 6mm diameter hole. The loss of initial gloss detected in Italy is higher than the one measured for the samples exposed in Holland and is due to the high level of air pollution caused by dust. It is possible that this kind of pollution, maybe due to the presence of steel-plants in the nearby, influenced the values measured, hiding the correlation between the loss of gloss and the color of the sample. This correlation can be seen in the values obtained for the samples exposed in Holland where there was less air pollution.

Results of the Dutch exposure (figure 2) show the effects of a marine environment through a high level of corrosion, on the cut edge and on the rivet, and a level of chalking higher than the one obtained after the exposure in Italy. In particular it was noticed that chalking happens with the same rate on all the samples, without correlation with the coating type.

File-cards of the degradation phenomena of coil coated steel sheet have been filled with the data obtained from these exposures. Anyway the application of the methodology drawn up by the research team at DISET and explained in Daniotti 1998 needed a deeper knowledge of degradation phenomena.

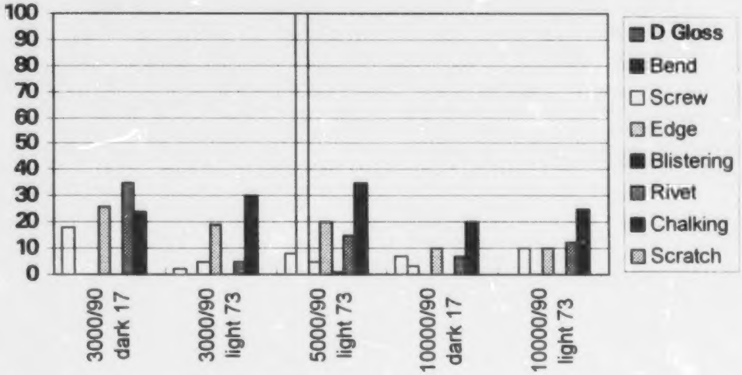

Figure 2: Results after 36 months of exposure in Holland

4 Results obtained from survey

Results obtained from the exposure tests in both locations were compared with the results from surveys on buildings built with the same coil coated steel sheet from 1985 to 1996. This comparison was made with the aim of obtaining an experimental validation of the mentioned filling-cards.

Relevant points were found for each building subject of survey. These points were characterized by particular exposure conditions, such as southerly exposure in places with a high sun radiation. Each point was recorded highlighting those particular aspects that may influence the natural aging of the building component, such as the distance from the sea, and reporting a brief description of the environment. The following parameters were recorded at intervals:

- color;
- gloss;
- chalking;
- dust retention.

Each point subject to survey is accompanied by a brief report, written on site, describing the maintenance state and highlighting, if any, external elements which may affect the maintenance state itself.

These surveys led to a substantial confirmation of data gained with the atmospheric environmental exposure tests, highlighting the action of sun radiation, which leads both to a discoloration even on light colored surfaces and, in association with other atmospheric agents, to chalking. The influence of air pollution and of particular kind of exposure (marine environment) is more evident in rust development. Anyway rust needs a propitious starting point such as a cutting edge or a screw hole to develop.

Fig. 3: Cremona - industrial district 1986

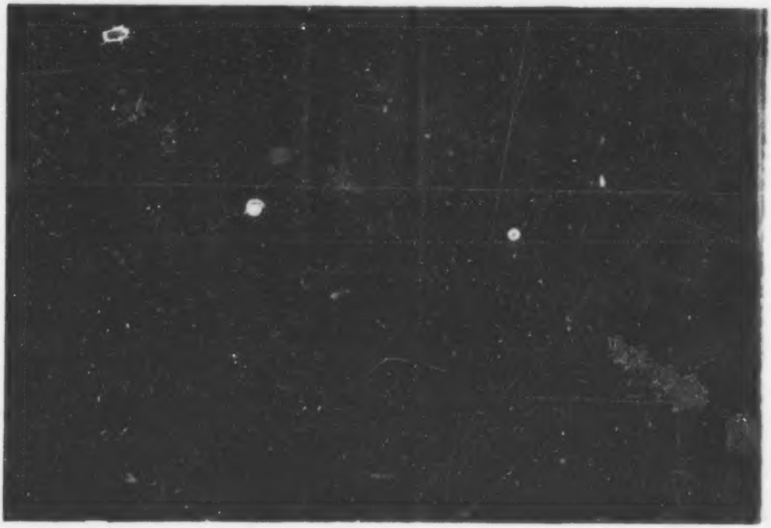

Fig. 4: Cremona - industrial district 1992

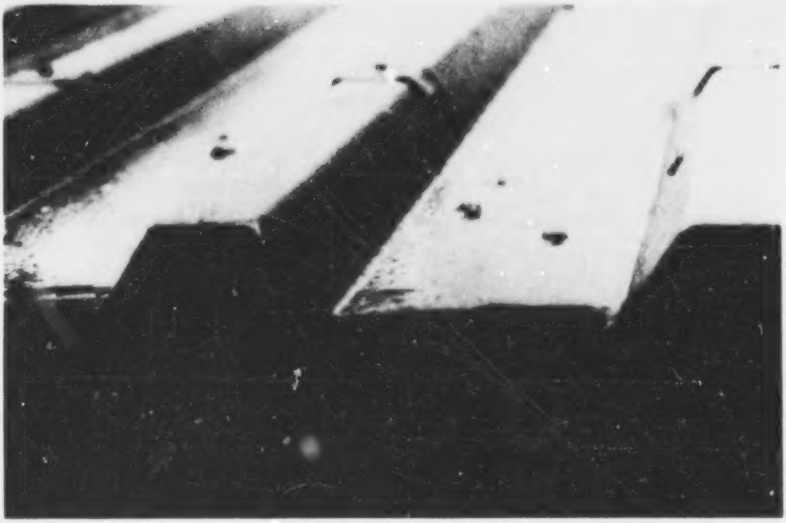

Fig. 5: Cutting edge

Fig. 6: Joint of four sheet

Examples of the results obtained from surveys are shown in figure 3, 4, 5, 6. Figure 3 and 4 shows two parts of a building, the first built in 1986 and the second built in 1992, both built using the same coil coated steel sheet. Figure 5 and 6 are two clear examples of starting point for dust.

5 Conclusion

Data collection leads to two main conclusions: first of all it is necessary to proceed to a higher correlation among atmospheric environment exposure data. For this reason the Italian site is already changing its equipment in order to conform the parameters of analysis to the Dutch ones. This will allow the researchers to better understand the influence of different climate. Secondly it will be interesting to compare data obtained during surveys with the ones obtained from atmospheric environment exposure. This comparison, if related with the climate, pollution and environmental data of every site, will allow the researchers to set a quantitative relation between natural aging on site and atmospheric environmental exposure aging. This relation will lead to a quicker and more reliable evaluation of coil coated steel sheet service life.

This experience stresses the necessity of atmospheric environmental exposure data and of surveys on existing buildings in order to obtain a better evaluation of building component service life. Another important result was to acknowledge that data from survey on existing building, if collected and processed in the right way, could be a guide for designers in the deep sea of design for durability.

6 References

1. Daniotti, B., Poli, T., Re Cecconi, F., Rigamonti, G.(1998), Methodology and experimental programmme to evaluate building components service life, in *Materials and technologies for sustainable construction*, Symposium A vol. 1, (ed. K. Fahlstedt), Gävle, pp.115.
2. Maggi, P.N. and others (1996) *Criteri di valutazione della propensione di durata in funzione degli andamenti qualitativi di variazione delle caratterisitche funzionali delle soluzioni tecniche*, Torino.
3. Rejna, M.G., Gottfried, A., Maggi, P.N., Ravetta, F. (1996) Method for reliability evaluation of building: application on a set of components, in *Application of the performance concept in building*, Vol. 2, (ed. R. Becker), Tel Aviv, pp. 6-93.

106 PERFORMANCE OF TAPE-BONDED SEAMS OF EPDM MEMBRANES: RESISTANCE TO PEEL-CREEP

W.J. Rossiter, Jr., M.G. Vangel, and K.M. Kraft

National Institute of Standards and Technology, Gaithersburg, USA

Abstract

This paper provides a summary of the major results of an industry-government consortium study on the creep resistance of tape-bonded seams of EPDM roof membranes. The study was undertaken to compare the creep-rupture performance of tape-bonded and liquid-adhesive-bonded seams of EPDM roof membranes and to recommend a protocol for evaluating the creep-rupture performance of such seams. The results showed that specimen sets fabricated using one of two tape-adhesive systems and cleaned, primed EPDM had mean peel times-to-failure that were generally comparable to, if not greater than, those of liquid-adhesive-bonded specimen sets. Based on the results, ASTM Technical Committee D08 on Roofing, Waterproofing, and Bituminous Materials issued "Standard Practice for Time-to-Failure (Creep-Rupture) of Adhesive Joints Fabricated from EPDM Roof Membrane Material."

Keywords: adhesive tapes, adhesive testing, building technology, creep resistance, EPDM roof membranes, roofing, seams, time-to-failure

1 Introduction

The use of pre-formed tapes for fabricating field seams of EPDM roof membranes has increased such that they are supplanting contact-type liquid adhesives for many applications. Use of tapes provides a reduction in volatile organic compounds (VOCs) released during seam fabrication, ease of application and decreased application time, and the availability of an adhesive system that has uniform properties such as width and thickness (Hatgas and Spector 1995). This paper

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summarizes the major results of a three-phase industry-government consortium study undertaken (1) to compare the creep-rupture performance of tape-bonded and liquid-adhesive-bonded seams of EPDM roof membranes, and (2) to recommend a protocol for evaluating the creep-rupture performance of such seams (Rossiter et al 1996, 1997, 1998a). The consortium study was initiated at the National Institute of Standards and Technology (NIST) in late 1994 in response to industry requests that independent evaluations be conducted and that nonproprietary data be developed on the performance of tape-bonded seams.

In the creep-rupture experiments, seam specimens of a fixed length are stressed under a constant load, and the time over which they sustain the load until total separation (i.e., the time-to-failure) is recorded. Results from previous NIST studies indicate that creep-rupture tests provide a sensitive procedure for evaluating the effects of a multiplicity of application and environmental factors for both tape-bonded and liquid-adhesive-bonded seams (Rossiter et al. 1996).

Limited field observations have suggested that many seam defects result from the rheological (deformation/flow) behavior of the adhesive and not chemical deterioration (Martin et al. 1991; Rossiter et al. 1994). Moreover, peel-type stresses are considered to be a major cause of seam defects, whereas lap-shear stresses are believed to have a minor effect on seam failures (Martin et al. 1991). Thus, the consortium study focused on characterizing the peel-creep resistance of seam specimens.

Since the mid-1980s, field experience with liquid-adhesive-bonded seams has been, in most cases, satisfactory. Consequently, the laboratory-measured times-to-failure of specimens prepared with cleaned EPDM and industry-recommended adhesive thickness (i.e., well-prepared specimens) have been taken as the benchmark for acceptable creep lifetimes. For this reason, a key objective of the study was a comparison between the creep performance of tape-bonded and liquid-adhesive-bonded specimen sets prepared under a variety of conditions.

2 Experimental

Two commercial tape systems comprised of a tape and primer (designated Tape System 1 or TS1, and Tape System 2 or TS2) and a commercial butyl-based liquid adhesive (designated LA) were used in fabricating seam specimens. All specimens described herein were prepared in the laboratory and had a minimum age of 28 days when tested. Specimen preparation and test procedures have been previously described (Rossiter et al. 1996, 1997, 1998a). A minimum of eight replicate specimens was included in any creep-rupture experiment. Note that results of statistical analyses conducted during the three phases are not presented in this paper; the reader is again referred to the original reports.

3 Factors affecting peel-creep response

3.1 Failure modes during peel-creep testing

The failure mode observed after creep-tests were conducted was generally cohesive (i.e., within the adhesive) with major exceptions as follows. In Phase I, TS2 specimens prepared with primer that was beyond its shelf life failed adhesively (i.e., at the interface of the adhesive and an adherend). In Phase II, specimens that were prepared using uncleaned and/or unprimed EPDM also failed adhesively. Finally, in Phase III-Task IV, TS1 specimens prepared at cold temperatures failed adhesively.

3.2 Phase I--effect of load

In Phase I, the creep-rupture response of tape-bonded and liquid-adhesive-bonded seam specimens were determined as a function of peel load. The peel-creep times-to-failure were determined under loads varying from 3.1 N to 24.9 N (0.7 lbf to 5.6 lbf) in increments of 3.1 N (0.7 lbf). The tests were conducted at room temperature, 23°C ± 2°C (73°F ± 4°F), in chambers having relative humidity between 40% and 45%.

Figure 1 is a plot of mean time-to-failure as a function of load for the 11 data sets included in the Phase I study. The plot characters represent the mean data points; whereas the curves represent the fit of the data to the model:

$$\ln(TTF) = b_0 + b_1 L + b_2 e^{b_3 L} \tag{1}$$

where TTF denotes the mean time-to-failure in hours, L is load in Newtons (or pounds force), and b_0, b_1, b_2, and b_3 are empirical constants. The function was fitted by nonlinear least squares methods; the estimated coefficients, standard deviations of the coefficients, and residual standard deviations have been previously reported (Rossiter et al. 1996). In all cases, the mean time-to-failure data points fall on or are close to the fitted curves. Observe in Figure 1 that the relationship between time-to-failure and load is relatively linear at the higher loads and nonlinear at lower loads. Note that no data points are included in this figure for tests conducted at 3.1 N (0.7 lbf), because no specimen failures were observed for this load after more than 16,800 h (about 23 months) of testing.

Figure 1 provides for a comparison of the times-to-failure of the tape-bonded and liquid-adhesive-bonded sample sets versus load. It is evident in Figure 1 that, with the exception of LA Replicate Set No. 1 at the lower loads, the times-to-failure for the tape-bonded sample sets were generally comparable to, or greater than, those of the liquid-adhesive-bonded sample sets. Figure 1 also provides an indication of the reproducibility of the time-to-failure data. It is apparent in Figure 1 that the liquid-adhesive-bonded sample sets were less reproducible than the two tape systems. At low loads (12.5 N or 2.8 lbf), the variability between the five LA replicate data sets was so wide that the minimum and maximum times-to-failure bracketed the times-to-failure of the tape-bonded sample sets. A consequence of this wide variability is that, under certain conditions, the liquid adhesive can provide seam sample sets which display substantially longer creep lifetimes than either other liquid-adhesive-bonded sample sets or tape-bonded sample sets. However, the conditions which produced the relatively long-lived LA Replicate Set No. 1 are not known and, hence, not predictably reproducible.

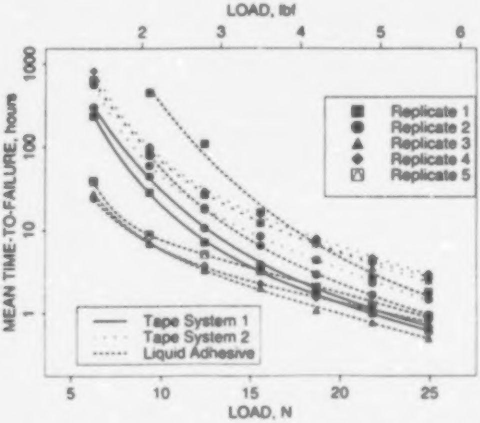

Fig. 1: Mean peel time-to-failure vs Load for the tape-bonded and liquid-adhesive-bonded specimens

In comparison to the liquid adhesive, the mean times-to-failure for the tape systems were more reproducible. For example, three of the four TS2 replicate sets had fitted curves that almost overlapped each other. Additionally, the two curves for TS1 were reasonably close to each other. This reproducibility may be because the tapes are factory-made (i.e., pre-formed) products and are not subject to some of the non-controllable application variables associated with the liquid adhesive.

3.3 Phase II--effect of material and application factors

The Phase II research was conducted to investigate the effects of material and application factors on peel creep-rupture response. The specimens were prepared under a variety of conditions according to a predetermined statistical design. The effects of two material factors, tape system and tape thickness, and five application factors, EPDM surface condition, primer, application temperature, application pressure, and time-at-temperature,[*] were examined. The peel-creep tests were conducted under a load of 9.3 N (2.1 lbf) at room temperature, 23°C ± 2°C (73°F ± 4°F), in chambers with relative humidity between 40% and 45%.

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[*] This refers to holding the completed specimen at the preparation temperature for a specified duration.

Two levels of each of the seven factors were included in the experimentation. The levels were chosen far enough apart so that the range of practical importance was generally covered:

- Tape system two commercial systems, TS1 and TS2
- Tape thickness thin, about 0.5 mm to 0.6 mm (0.020 in to 0.025 in), and 'standard,' about 0.9 mm to 1.0 mm (0.035 in to 0.040 in)[**]
- EPDM surface condition clean and contaminated
- Primer primed EPDM and unprimed EPDM
- Application Temperature low, 5 °C (41 °F), and high, 60 °C (140 °F)
- Application Pressure low, 0.2 MPa (30 lbf/in^2), and high, 2 MPa (300 lbf/in^2)
- Time-at-Temperature short, about 20 h to 24 h, and long, about 672 h to 960 h

A full factorial design in Phase II was beyond the scope of the study, so a half-fraction of the full factorial design was chosen. This design included the four combinations of material factors, and 16 of the 32 possible combinations of the application factors (Rossiter et al. 1997). The same 16 combinations of the application factors were assigned to each of four combinations of the material factors. It was considered that this design would lead to the selection of combinations of levels of application factors which produced good seams, as quantified by time-to-failure, for *all* combinations of material factors.

Figure 2 summarizes the Phase II mean time-to-failure data for each of the 16 combinations of application factors. The plot characters (along with the figure legends) identify the combinations of the tape system and tape thickness (i.e., the material factors). The horizontal axis specifies the levels of the five application factors. The application factor combinations are ordered in increasing mean response. The mean for the four sample sets (i.e., TS1-thin, TS1-standard, TS2-thin, and TS2-standard) prepared at each combination of application factors is indicated by the dotted line.

Figure 2 also contains a horizontal dashed line. It represents the average time-to failure for the three sets of well-prepared liquid-adhesive-bonded specimens (i.e., those fabricated with industry-recommended adhesive thickness and clean EPDM) that had the lowest average times-to-failure at 9.3 N (2.1 lbf) in Phase I (fig. 1). The dashed line is included to provide a point of comparison between the Phase II data for tape-bonded sample sets and liquid-adhesive-bonded sample sets. The lowest mean times-to-failure for the liquid-adhesive-bonded sample sets were selected for comparison, because sets with these mean times-to-failure are considered representative of the minimum of the range of liquid-adhesive-bonded seams typically used in current practice. That is, the question addressed is whether tape-bonded sample sets prepared under a variety of conditions perform equal to, or better than, the lowest mean performance of well-prepared liquid-adhesive-bonded sample sets.

Figure 2 provides a basis for comparing the effects of varying both the material and application factors on time-to-failure. Regarding material factors, the "standard" thickness sample sets had, in most cases, greater times-to-failure than the thin sample

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[2] The thicker tapes had thicknesses typical of those commercially available at the time of the study; thus, they were designated as having 'standard' thickness.

sets. Simultaneous confidence intervals (details not shown) demonstrated that the "standard" thickness sample sets had significantly (95 % confidence level) longer mean times-to-failure than the corresponding thin sample sets for 11 of the 16 application factor combinations. Also, the thin sample sets did not have significantly longer times-to-failure in the remaining five cases. That is, the time-to-failure results provided no evidence that thinner tapes should be used.

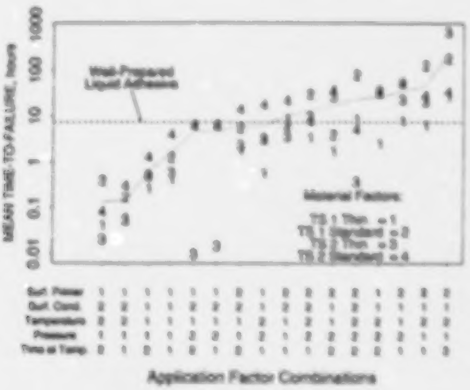

Fig. 2: Mean peel time-to-failure versus combinations of application factors. Primer: Unprimed = 1, Primed = 2; EPDM Surface Condition: Clean = 1, Contaminated = 2; Application Temperature: Low = 1, High = 2; Application Pressure: Low = 1, High = 2; Time-at-Temperature: Short = 1, Long = 2.

The horizontal dashed line represents the mean time-to-failure of the three sets of well prepared, liquid-adhesive-bonded specimens (i.e., those fabricated with industry-recommended adhesive thickness and clean EPDM) that had the lowest mean times-to-failure at the 9.3 N (2.1 lbf) load in Phase I (see fig. 1). The dotted line represents the mean for the four sample sets of tape-bonded specimens prepared at each combination of application factors.

It may also be seen in examining Figure 2 that, for the combinations of application factors, no trend of time-to-failure behavior as a function of tape system is evident. For 10 of the 16 combinations of application factors, simultaneous confidence intervals showed no significant difference in the times-to-failure of the TS1 and TS2 data sets. Where a significant difference existed, the TS1 sample sets had greater mean time-to-failure in four cases, while the TS2 sample sets had greater mean time-to-failure in two cases.

Regarding application factors, it was of interest to compare in Figure 2 the time-to-failure data for the 16 Phase II sample set combinations with the average time-to-failure for the three sets of liquid-adhesive-bonded specimens from Phase I. In considering the left side of Figure 2, it was found that eight of the application factor combinations had significantly lower, or at best comparable, mean times-to-failure versus the average for liquid-adhesive-bonded sample sets. Of these eight application factor combinations, seven sets were unprimed. Moreover, of those seven unprimed sets, four included the contaminated EPDM surface condition. On the right side of Figure 2 among the six application factor combinations having the highest mean times-to-failure, five had primed EPDM surfaces and five had clean EPDM surfaces. Moreover, these six combinations having the highest mean times-to-failure included the four cases where the EPDM was both primed and cleaned. That is, in these four cases, the mean times-to-failure of the tape-bonded sample sets were comparable to, if not significantly greater than, that of the liquid-adhesive-bonded sample sets. A corollary to this latter observation was that, within the limits set in the experiment, temperature, pressure, and time-at-temperature did not affect the creep-rupture response of the primed and clean sample sets to the extent that the mean times-to-failure were less than that of the liquid-adhesive-bonded sample sets.

The findings in Figure 2 quantify the importance of both priming and cleaning the EPDM rubber, which is in accord with manufacturers recommendations for fabrication of tape-bonded seams. In general terms, the data in Figure 2 suggest that, if tape-bonded seams are primed, it is reasonable to expect that they will have creep lifetimes comparable to liquid-adhesive-bonded seams. On the other hand, primed tape-bonded seams having clean EPDM can have longer creep lives than liquid-adhesive-bonded seams. And, it is important to point out, the relatively long creep lifetimes of seams made with primed, clean EPDM are not expected to be adversely affected by application temperature and pressure (within the limits selected for the study), which are factors that are uncontrollable in practice.

3.4 Phase III-Task I--effect of elevated temperature during creep testing

The Phase III-Task I investigation was conducted to compare the peel-creep performance of tape-bonded and liquid-adhesive-bonded sample sets tested at the elevated temperatures. Peel creep-rupture tests were performed at 23°C, 40°C, and 70°C (73°F, 104°F and 158°F) under each of three loads, 9.3 N, 12.5 N, and 15.6 N (2.1 lbf, 2.8 lbf, and 3.5 lbf). Additionally, at 70°C (158°F), a creep test was performed under a 6.2 N (1.4 lbf) load.

Figure 3 (A, B, and C) is a plot of the geometric mean time-to-failure results for the three adhesive systems. Polynomial models with load and temperature as variables were fitted to summarize the time-to-failure results. The models were fitted separately to data for the three adhesive systems. The simplest model fitting the time-to-failure (TTF) data was:

$$\log(\mathrm{TTF}) = C_0 + C_1L + C_2L^2 + C_3T + C_4T^2 + C_5TL \qquad (2)$$

where L denotes load, T is temperature, and C_0 through C_5 are coefficients (see Rossiter et al. 1998a) estimated by least squares. The fitting of this model was used to produce the curves and error bars in Figure 3. The model fitted the data well, as evidenced by the closeness of the data points to the curves. As evidenced in Figure 3, for the three adhesive systems, the mean time-to-failure decreased with an increase in temperature and load, although the decrease was slight for TS2 between 23°C and 40°C (73°F and 104°F).

From Figure 3, the following observations comparing the Phase III-Task I peel creep-rupture results for the tape-bonded and liquid-adhesive-bonded sample sets may be made. For any combination of test temperature and load, the times-to-failure of TS1 and TS2 were greater than that of LA. The differences were statistically significant. When temperature increased from 23°C to 40°C (73°F to 104°F) for a given load, TS2 showed less decrease in time-to-failure than LA; TS1 and LA had about the same decrease. When temperature further increased from 40°C to 70°C (104°F to 158°F) for a given load, both TS1 and TS2 experienced less decrease in time-to-failure than LA.

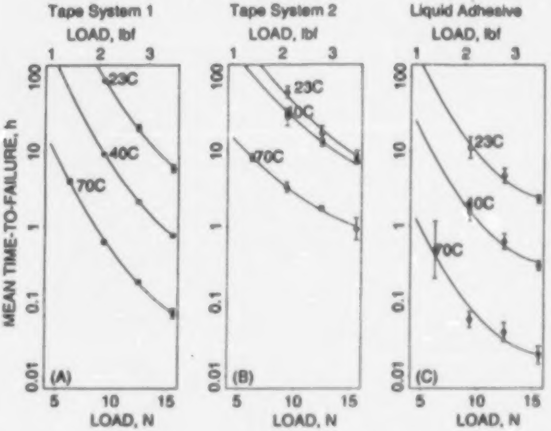

Fig. 3:Mean peel time-to-failure as a function of temperature and load for each adhesive system. The uncertainty bars represent 95% confidence intervals on the means

3.5 Phase III-Task II--effect of elevated temperature exposure prior to testing

The Phase III-Task II investigation was conducted to compare the peel-creep performance of tape-bonded and liquid-adhesive-bonded sample sets after heat exposure. Tape-bonded and liquid-adhesive-bonded samples were heated in forced air ovens at temperatures of 60°C, 75°C, and 90°C (140°F, 167°F, and 194°F) for 30 days, 90 days, and 150 days (1 day). After exposure, peel time-to-failure was measured at 23°C (73°F) under a creep load of 9.3 N (2.1 lbf). Peel time-to-failure was also determined for room temperature control (i.e., non-heat exposed) sample sets.

Figure 4 is a plot of geometric mean peel time-to-failure for the three adhesive systems as a function of heat-exposure time and temperature. The open plot character represents controls; the closed plot character represents heat-exposed specimens. From Figure 4, the following observations comparing the Phase III-Task II peel creep-rupture results for the tape-bonded and liquid-adhesive-bonded sample sets may be made. For LA with respect to the control, mean times-to-failure generally were unaffected or increased due to the heat exposures. In comparison, TS1 mean times-to-failure increased, and the increases were greater for TS1 than for LA. For example, for the exposure of 90 days at 75°C (167°F), the TS1 exposed sample set had a mean time-to-failure that was estimated to be about 100 times greater than the control; in contrast, the mean time-to-failure of the

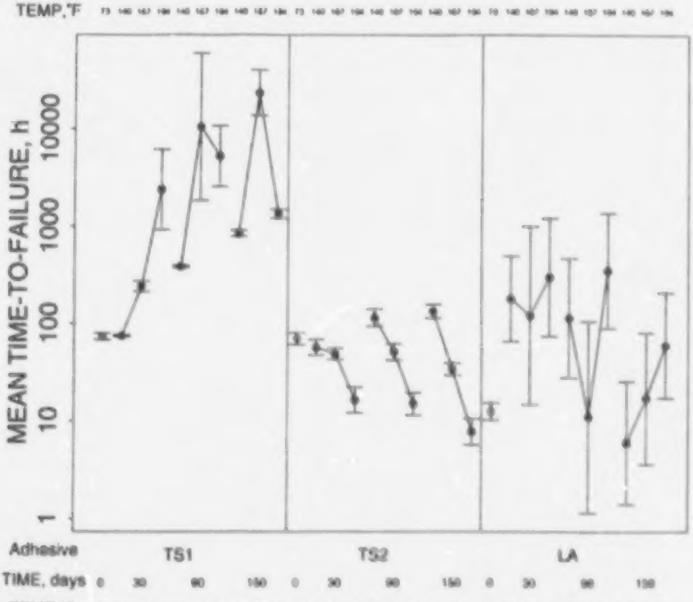

Fig. 4: **Mean peel time-to-failure for the three adhesive systems as a function of heat-exposure time and temperature. The uncertainty bars represent 95% confidence intervals on the means. The open plot characters represent data for unexposed controls**

LA exposed sample set was about 4 times greater than its control. In comparison to LA behavior, TS2 mean times-to-failure were unaffected or decreased upon heat exposure. The greatest decrease was observed for the 150-day exposure at 90°C (194°F). In this case, the exposed sample set had a mean value of 8.3 h, which was about 8.5 times less than the control. The 8.3 h time-to-failure was slightly less than the 13.1 h mean time-to-failure of the LA control; the difference was significant.

It was of interest to compare the Phase III-Task II time-to-failure results with data determined for field seams. In Phase III-Task II, many TS1 heat exposed specimens were found, vis-a-vis the control, to be very creep resistant with estimated times-to-failure greater than 10,000 h (fig 4). The shortest mean time-to-failure for a TS2 heat exposed sample set was about 8 h. These extreme values may be compared with the time-to-failure results obtained from field seams that failed cohesively during creep testing. These seams were sampled from 10 roofs having ages from 0.1 year (i.e., an essentially new roof) to 3 years, and had performed well during their relatively short time in service (Rossiter et al. 1998b). When tested in the laboratory, these field seams had mean times-to-failure ranging from 10.9 h to 173.7 h. Thus, in this comparison, no evidence is found that the long times-to-failure produced in the laboratory upon heat exposure have occurred in the field. The comparison also indicates that, although some tape-bonded laboratory samples underwent a decrease in time-to-failure upon heat aging, the resultant values were not appreciably less than mean times-to-failure that have been obtained for field seams.

3.6 Phase III-Task III--effect of exposure to industry-developed protocols

The Phase III-Task III investigation was conducted to compare the peel-creep performance of tape-bonded and liquid-adhesive-bonded sample sets after laboratory exposure to the protocols developed by RMA (1989) and SPRI (1995). Both protocols subject specimens to dry heat in an oven, heat in water, and freeze-thaw cycling. In contrast to the RMA protocol, the SPRI protocol requires that the specimens be elongated in shear at 20% while being subjected to the freeze-thaw cycling and to heat cycling (conducted in an ultraviolet test chamber). The RMA protocol stipulates cycling for 28 days with an option to conduct it for 56 days. In the Phase III-Task III experiment, samples were cycled for 28 days and for 56 days so that data would be available on an RMA exposure period that was the same length as the SPRI exposure period, which is stipulated to be 56 days. After the RMA and SPRI exposures were completed, peel times-to-failure of the tape-bonded and liquid-adhesive-bonded specimens were determined at 23°C (73°F). The creep load was 9.3 N (2.1 lbf).

Figure 5 is a plot of geometric mean peel time-to-failure after subjecting specimens of each adhesive system to the industry exposure protocols. Data on room temperature control specimens are included. Figure 5 provides a comparison of the Phase III-Task III peel creep-rupture results between the tape-bonded and liquid-adhesive-bonded sample sets. With regard to the RMA protocol, LA mean time-to-failure increased upon exposure. In comparison, TS1 and TS2 had increased and decreased time-to-failure, respectively. TS2 mean time-to-failure after 56-day exposure was not statistically different than that of the LA control. In the case of the SPRI protocol, the LA mean time-to-failure increased upon exposure. In comparison, TS1 was unchanged, and TS2 decreased. Again, as was the case for the RMA exposure, TS2 mean time-to-failure after 56-day exposure was not statistically different than that of the LA control.

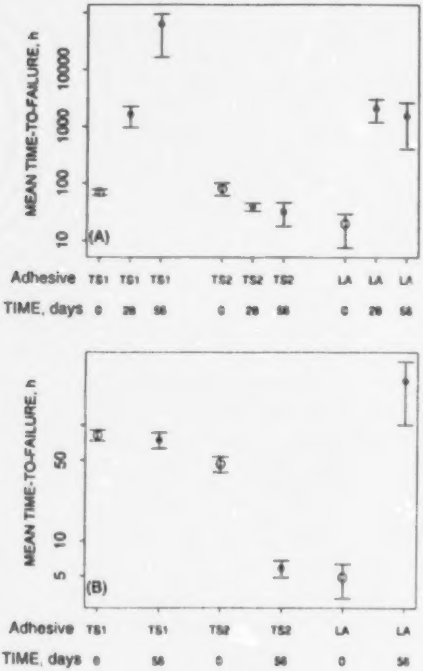

Fig. 5: Mean peel time-to-failure after subjecting specimens to the: (A) RMA exposure protocol for 28 days and 56 days, and (B) SPRI exposure protocol for 56 days. The uncertainty bars represent 95% confidence intervals on the means. The open plot characters represent data for unexposed controls

It was of interest to compare the Phase III-Task III peel time-to-failure results with data determined for tape-bonded field seams. As previously indicated, mean times-to-failure of tape-bonded seams that performed well in service and failed cohesively in creep tests ranged from 10.9 h to 173.7 h (Rossiter et al. 1998b). In comparison, for TS1 after SPRI exposure, the mean time-to-failure was 75 h, i.e., in the middle of the field-seam range. After 56-day RMA exposure which considerably increased the creep resistance of the TS1 laboratory sample sets, the minimum mean time-to-failure was estimated to be above 10,000 h. This value was substantially beyond the field-seam range. That is, the comparison indicates that the relatively long TS1 times-to-failure, finding parallels that of Phase III-Task II, wherein it was found that prolonged times-to-failure due to laboratory heat exposure have not been evidenced for field seams. Comparisons of the TS2 results with those of field seams indicate that the mean times-to-failure of the RMA and SPRI sample sets (about 33 h and 6 h, respectively) were comparable to the shorter times-to-failure determined for field seams.

3.7 Phase III-Task IV--effect of cold temperature preparation

The Phase III-Task IV investigation was conducted to compare the creep-rupture performance of tape-bonded and liquid-adhesive-bonded specimens after preparation at cold temperatures. Tape-bonded and liquid-adhesive-bonded peel samples were prepared in a temperature-humidity controlled glove box at temperatures of -5°C to -2°C (23°F to 28°F), and –1°C to 1°C (30°F to 34°F) with relative humidity between 10% and 15%. The samples were kept at about -4°C (25°F) for 28 days after preparation, and then shipped at ambient temperatures overnight to NIST. Room-temperature laboratory specimens, i.e., controls, were prepared at 21°C to 23°C (69°F to 73°F) with a relative humidity of 45% to 50%, and kept under those conditions for a minimum of 28 days before testing. After receipt at NIST, the cold-temperature samples were placed in a freezer at about -20°C (-4°F), where they were stored until warming to room temperature for specimen preparation and peel creep-rupture testing. The time elapsed between removal from the freezer and placement under testing did not exceed 24 h.

Figure 6 is a plot of geometric mean time-to-failure for the cold temperature prepared specimen sets. From Figure 6, the following observations comparing the Phase III-Task IV peel creep-rupture results for the tape-bonded and liquid-adhesive-bonded systems are noted. LA mean times-to-failure decreased upon cold temperature preparation. In comparison, TS1 mean time-to-failure also decreased, while that of TS2 was unaffected. The reduction in mean time-to-failure due to cold temperature preparation was greater for TS1 than for LA. The mean time-to-failure of the -4°C (25°F) TS1 sample set was a factor of approximately 28 less than that of its control, whereas the mean times-to-failure for the two LA sample sets differed by a factor of only 2.3. Nevertheless, although TS1 was more affected than LA by the cold temperature preparation, the mean time-to-failure of the -4°C (25°F) TS1 sample set was still greater than that of the corresponding LA sample set. The reason was that the mean time-to-failure of the TS1 control was considerably greater than that of the LA control.

4 Summary and conclusions

The use of tape-adhesive systems for fabricating field seams of EPDM roof membranes has significantly increased in the 1990s. A three-phase industry-government consortium study has been conducted to develop nonproprietary data on tape-bonded seam performance. In Phase I, the creep-rupture response (time-to-failure) of tape-bonded seam samples subjected to various peel loads under ambient conditions was compared to that of liquid-adhesive-bonded samples. In Phase II, two material factors (tape system and tape thickness) and five application factors (EPDM surface condition, (tape system and tape thickness) and five application factors (EPDM surface condition, primer, application temperature, application pressure, and time-at-application temperature) were investigated in a statistically designed experiment for their effects on peel creep-rupture response. In Phase III, four tasks investigated the effects of: (I) elevated test temperatures, (II) elevated temperature exposure prior to loading, (III) exposure to industry-developed protocols (i.e., Rubber Manufacturers Association and SPRI), and (IV) cold temperature preparation on peel creep-rupture response.

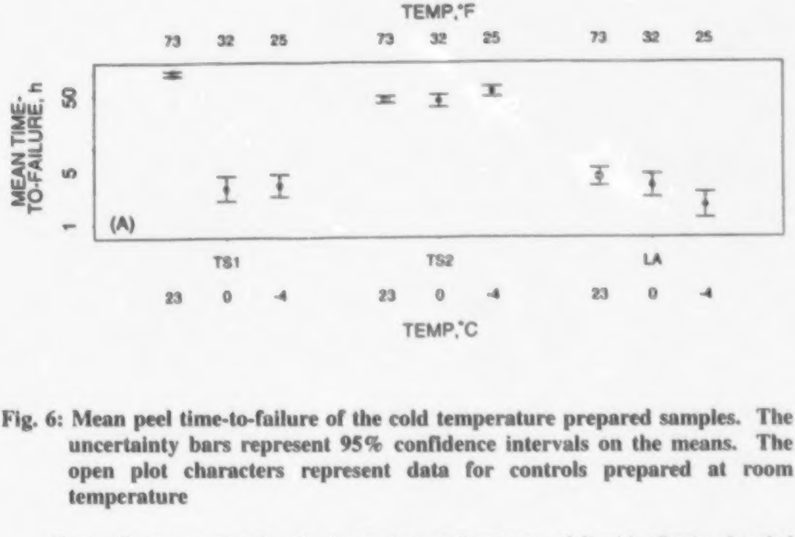

Fig. 6: Mean peel time-to-failure of the cold temperature prepared samples. The uncertainty bars represent 95% confidence intervals on the means. The open plot characters represent data for controls prepared at room temperature

For well over a decade, the in-service performance of liquid-adhesive-bonded seams has been generally satisfactory. Consequently, their creep-rupture performance has been the benchmark against which the creep performance of tape-bonded seams may be judged. Consistently throughout this three-phase study, specimen sets fabricated using either one of two tape-adhesive systems and cleaned, primed EPDM had mean peel times-to-failure that were generally comparable to, if not greater than, those of liquid-adhesive-bonded specimen sets. Moreover, although some laboratory exposures brought about decreases in mean peel times-to-failure for some tape-bonded seams, the resultant times-to-failure were yet comparable to values measured for some field-sampled tape-bonded seams that had performed well in service. Based on the results of the study, ASTM Technical Committee D08 on Roofing, Waterproofing, and Bituminous Materials issued "Standard Practice for Time-to-Failure (Creep-Rupture) of Adhesive Joints Fabricated from EPDM Roof Membrane Material." Consistent with a main objective of the study, this ASTM Standard provides a protocol for evaluating the creep-rupture performance of EPDM seam specimens regardless of the adhesive system used in their fabrication.

5 Acknowledgements

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6 References

Hatgas, D.J. and Spector, R.C. (1995) Tape products used in EPDM roofing systems. *Proceedings, ACS Rubber Division Symposium*, American Chemical Society, Philadelphia, PA, 26 pages.

Martin, J.W., Rossiter, W.J., Jr., and Embree, E. (1991) Factors affecting the strength and creep-rupture properties of EPDM joints. *Proceedings, Third International Symposium on Roofing Technology*, U.S. National Roofing Contractors Association, Rosemont, IL, pp. 63-71.

RMA (1989) Minimum peel strength requirements for adhesives used in seaming black EPDM membranes. RP-10, Rubber Manufacturers Association, Washington, DC, 3 pages.

Rossiter, W.J., Jr., Martin, J.W., Lechner, J.A., Embree, E., and Seiler, J.F., Jr. (1994) Effect of adhesive thickness and surface cleanness on creep-rupture performance of EPDM peel and lap-shear joints. *Roofing Research and Standards Development: 3rd Volume*, ASTM STP 1224, American Society for Testing and Materials, West Conshohocken, PA, pp. 123-138.

Rossiter, W.J., Jr., Vangel, M.G., Embree, E., Kraft, K.M., and Seiler, J.F., Jr. (1996) Performance of tape-bonded seams of EPDM membranes: comparison of the peel creep-rupture response of tape-bonded and liquid-adhesive-bonded seams. *Building Science Series 175*, National Institute of Standards and Technology, 73 pages.

Rossiter, W.J., Jr., Vangel, M.G., Kraft, K.M., and Filliben, J.J. (1997) Performance of tape-bonded seams of EPDM membranes: effect of material and application factors on peel creep-rupture response. *Building Science Series 176*, National Institute of Standards and Technology, 61 pages.

Rossiter, W.J., Jr., Vangel, M.G., and Kraft, K.M. (1998a) Performance of tape-bonded seams of EPDM membranes: factors affecting the creep-rupture response of tape-bonded and liquid-adhesive-bonded seams. *Building Science Series 177*, National Institute of Standards and Technology, 55 pages.

Rossiter, W.J., Jr., Vangel, M.G., and Kraft, K.M. (1998b) Performance of tape-bonded seams of EPDM membranes: a field investigation. *Proceedings*, Xth Congress of the International Waterproofing Association Held in Copenhagen, International Waterproofing Association, Basford, Nottingham, England.

SPRI (1995) Recommended laboratory test methods for liquid and tape adhesives used to splice single ply membranes, SPRI, Needham, MA, 32 pages.

107 IMPROVING THE DURABILITY OF FLAT ROOF CONSTRUCTIONS

C. Rudbeck and S. Svendsen

Technical University of Denmark, Lyngby, Denmark

Abstract

Flat roof constructions are mainly used on commercial, institutional and industrial buildings, where insulation is placed on top of the load-bearing deck and then covered with a roof membrane. Through time, there is a risk that the membrane will allow water passage as holes might form due to weathering effects or physical loads. Water will then enter the insulation, and as a vapor retarder is normally found below the insulation thus trapping the water in the insulation, the leak can remain undetected for a long period. When the leak is finally discovered, the insulation has to be discharged as there is no easy method of drying it.

To be able to dry the insulation, and thereby regain the functional requirements of the roofing system, two new solutions for insulating flat roofs with existing materials are proposed for high density mineral wool and expanded polystyrene. Monitoring equipment are part of the system, thereby making it easier to detect leaks faster. When a leak is detected, the membrane is repaired locally. In order to remove water which has already entered the insulation, an air gap or a system of air channels between the deck and the insulation is subjected to forced ventilation with outdoor air. When the water is removed, the ventilation is stopped, and the roofing construction can continue to function as intended.

Roofing systems where trapped moisture can be removed are cost-effective compared to traditional roofing insulation systems, and as leakage can be treated, they have a longer life span reducing the overall cost. Furthermore systems, where moisture can be removed, offer a high probability that the thermal conductivity remains at its designed value through the entire life of the roofing system. If the roofing membrane should fail, the insulation can be dried and the thermal conductivity will return to its original value instead of a much higher value found in traditionally insulated constructions.

Keywords: flat roof, moisture, durability, cost-effective, ventilated air gap

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1 Introduction

Flat roof constructions are commonly used throughout a majority of the industrialized world, but are often suffering from excessive moisture content and therefore in need of renovation. In recent years, the reputation of flat roofs has changed to the poorer, changing a large portion of roofs on small buildings from flat to sloped roofs. Commercial buildings, storage buildings etc., with large roof areas are still being renovated, but not with sloped roofs as these are difficult to construct on large buildings.

Problems in roofs related to moisture are seldom caused by vapor transport from the building below. Instead water, which is either trapped during the construction phase or originating from rain ingress through leaks, plays a major role in the deterioration process. As the membranes on each side of the insulation are very tight, water is trapped inside the insulation. The result is a higher energy demand for the building below as the thermal conductivity of the insulation is increased (ISO 1997). In light weight structures the energy demand can increase even more as latent heat transfer plays an important role (Rode and Rudbeck 1998). Latent heat can increase the annual heat demand by up to 15% in some locations by having just 0.5% (mass) water content in the insulation. As water can accumulate in the roof construction without the occupants' knowledge, it might take several years before leaks are discovered and renovation of the roof construction is initiated.

Another approach to incoorporate drying capabilities in a roof construction is by using a water permeable vapor retarder (IEA 1996a). Water is then transported from the insulation and through the vapor retarder. However, a disadvantage with this approach is that the first sign of a leaky roof membrane is when water is dripping from the ceiling in the room below.

The straightforward solution is to repair the leak in the roof membrane and dry the insulation. However, as the insulation lies between two tight membranes, this solution is not possible. Instead renovation is performed either by adding a new layer of insulation and a new roof membrane on top of the old roof membrane, or by removing the old roof membrane and insulation and replace it with a new roof membrane and insulation. Both of these methods cure the symptoms of the roof, but do not take care of the illness as the new roof will fail in the same manner after some years. In order to solve the problem there is a need for new techniques for insulation of roofs. If the insulation could be dried after a leak has been detected and repaired, the service life of the roof construction could be extended, which would improve the total economy of the roofing system. The fundamental principle of flat roof constructions is that they should be constructed acknowledging that they will leak at some point in time. Whereas we currently construct roofs with a high safety against leaks we need to combine it with the roofs being prepared for reparations and service.

3 Development of roofing systems with improved durability

To improve the durability of roofing systems, new designs are suggested where moisture can be removed from the insulation in case of built-in moisture or leaks in the roof membrane.

3.1 Roofing insulation based on expanded polystyrene (EPS)

The roofing system consists of a vapor retarder with an imbedded moisture sensor, EPS insulation panels and a roofing membrane. To obtain a sloped roof, the thickness of the insulation panels is varied. During the production of the insulation panels, grooves are created in the down facing side of the insulation. The grooves are created in the lowest 60 mm slab of the insulation. The downside of the insulation is shown on figure 1.

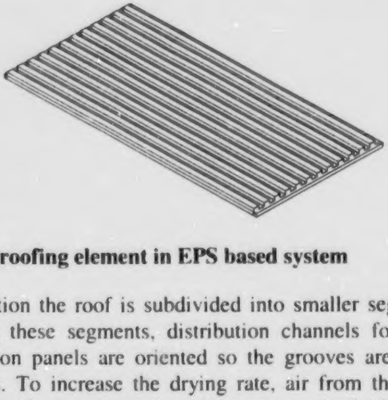

Fig. 1: Downside of roofing element in EPS based system

During installation the roof is subdivided into smaller segments, e.g. 10 x 10 meters. Surrounding these segments, distribution channels for ventilation air are created. The insulation panels are oriented so the grooves are connected with the distribution channels. To increase the drying rate, air from the outdoor climate is forced through the distribution channels and the grooves in the insulation panels. The air flow is supplied by a ventilation unit which is installed in the segment where the high water content is detected. During the construction of the roofing system, vertical ducts for the ventilators are established in the insulation. These ducts are sealed at the top until needed. An overview from the downside of the insulation system, showing the grooves, the distribution channels and ventilation units is shown on figure 2.

Based on information from the imbedded moisture sensors, the leak in the roof membrane is found and repaired. A ventilation unit in placed is one corner of the segment containing water while the air inlet is placed in the diagonal corner. When the moisture content in the insulation is sufficiently low, the ventilation unit is removed and the caps at the the air inlet and outlet is closed. An air inlet with a closed cap is shown on figure 3. The air inlet is a pipe (100 mm in diameter) which is placed in a 60 x 60 cm insulation element during the construction. The grooves which are created in the insulation element is shown in detail on figure 2. Besides drying the insulation in case of leaks the method can also be used as a moisture control as the insulation can be dried once in a while.

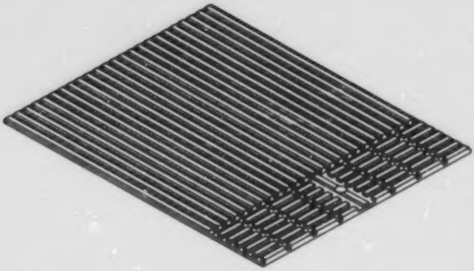

Fig. 2: Downside of roofing insulation based on expanded polystyrene

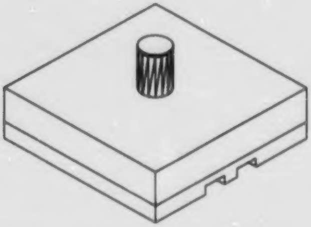

Fig. 3: Air inlet with closed cap for EPS based roofing insulation system

3.2 Roofing system based on high-density mineral wool

Instead of using EPS insulation as described in section 3.1, high density mineral wool can be used in a similar roofing system. As mineral wool is more permeable than EPS insulation, moisture transport is faster, making it possible to enlarge the distance between the grooves while obtaining the same drying rate.

3.3 Moisture indicator for roofing systems

An important property for a moisture indicator in a roofing system is to signal when and where moisture has entered the roof construction. Inspection is not a good solution as it is time consuming and not continous.

As an alternative, moisture sensors can be embedded in the vapor retarder, which is normally placed between the deck and the insulation system. Due to gravity, some of the rain which penetrates the roof membrane will locate itself on the vapor retarder as the insulation material is very water permeable. If water is detected on the vapor retarder, the building client should be alarmed in order to have the problem fixed.

The moisture indicator is based on measurements of resistance between two thin metal wires. Besides detecting if excessive moisture is present, the moisture indicator can also give a rough estimate of the location of the water. Production of the moisture indicators is done in 10 cm wide bands, and these can be placed in a regular pattern across the roof or at places of special interest, e.g. near gutters.

4 Results from simulations of roofing systems

The performance of the roofing systems in comparison with traditional roofing systems was examined by a series of simulations and calculations.

4.1 Roofing system based on expanded polystyrene

By creating grooves in the insulation panels, which are used when the roof should be dried, the thermal property of the roofing system is changed. Even though the grooves are unventilated, the thermal resistance is lower than that of still air, mainly due to convection and thermal radiation in the groove. A calculation of the thermal resistance for the insulation layer followed by a comparison with similar values for traditional insulation systems are shown in table 1. The dimension of one groove is 30x50 mm (height x width) and these are established for every 100 mm in the insulation panels width. For traditional roofing systems, two values are given for the thermal resistance of the insulation layer. One where the insulation is dry (thermal conductivity 0.039 W/mK) and one with 10% dry weight water content in the insulation (thermal conductivity 0.050 W/mK according to (ISO 1997)). In the calculation of thermal resistance for the traditional roofing system the insulation thickness is 195 mm which is the average insulation thickness used according to the Danish Building Code (1995). As insulation material is removed to create the grooves in the new roofing system, the thermal resistance should be lower. However, it is possible to, during the fabrication process, add the removed insulation at the top of the insulation layer. This increases the insulation thickness to 210 mm without using extra material and without waste of raw material.

Table 1: Thermal resistance of insulation layer in different roofing systems

Roofing system	Thermal resistance (m^2K/W)
Traditional roofing system (dry insulation)	5.0
Traditional roofing system (wet insulation)	3.9
New roofing system	5.0

By creating grooves in the insulation layer the thermal resistance of the new insulation system is equal to the reference case with dry insulation, but higher than the reference case with wet insulation. Moisture content in traditional roofing systems might even be higher than estimated in these calculations, and the thermal resistance will then be even lower.

To have a high drying rate for the insulation system, a good air distribution is needed. Due to the large dimensions of the distribution channels, the pressure drop is low and air is distributed well. From the air intake through the distribution channels and grooves to the fan, the total air-pressure difference is estimated to be below 50 Pa. Such an over-pressure can easily be supplied by a standard roof ventilation unit which can supply the needed air flow.

The drying rate of the system, compared with a traditional roofing system based on EPS, was examined by a series of simulations using a heat and moisture transfer model (Pedersen 1990) which has been validated in e.g. (IEA 1996). The constructions consist of modified bitumen as roof membrane, 195 mm EPS insulation and a 0.15 mm PE vapor retarder on top of the supporting concrete deck. Initially the average water content in the EPS was 10 mass-% to simulate water having penetrated the roof membrane. The exact water distribution through the EPS was decided by transferring results from a series of simulations on a model consisting of a layer of EPS located between two extremely vapor-tight surfaces. Indoor temperature conditions for both models ranged from 21°C to 23°C and the relative humidity was between 40% and 60%. As outdoor conditions the Danish Test Reference Year (Lund 1985) was used. Simulations started on 1st January year 1 and continued until 31th December year 3. Forced ventilation was performed through the grooves at the bottom of the EPS layer with an average air velocity of 0.2 m/s. The average air velocity in the distribution channels during the simulation period was 1.5 m/s. The forced ventilation was stopped after 60 days.

The effect of drying the insulation by ventilation can be seen on figure 5 with the average moisture content in the EPS insulation for a reference case and the newly developed system.

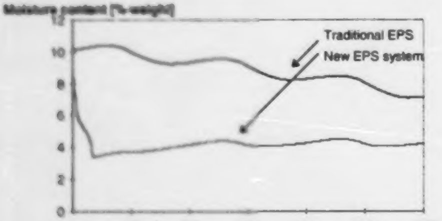

Fig. 4: Average moisture content in EPS in reference roof and new system

During the first 60 days the new EPS system is dried out due to the forced ventilation at the buttom of the insulation layer. As the drying is stopped, the moisture content in the insulation starts to increase due to diffusion of water vapor from the room below. After some years the moisture content in the insulation will be considered too high and drying will be initiated. Figure 4 also shows that the moisture content decreases in the traditional EPS insulation system. which is due to diffusion through the vapor retarder and the roofing membrane. However, this is under the assumption that no leaks develope in either the vapor retarder or the roof membrane.

Unfortunately, no measurements have been performed in the laboratory to validate these theoretical calculations. Such measurements are expected to be initiated in the near future.

4.2 Roofing system based on high-density mineral wool

The principles in the roofing system based on high-density mineral wool are the same as the roofing system based on expanded polystyrene. As the permeability is higher for mineral wool than for EPS, moisture transport is faster and it is therefore expected that an insulation system based on the described principles with high-density mineral wool will have a higher drying rate.

5 Discussion of results

From a theoretical point of view both the system which is based on grooves in the insulation and the system based on roofing cassettes seem functional. Results from calculations of drying for both systems show that drying is found, and therefore that the insulation can return to its initial dry state making an expensive replacement of the insulation unnecessary.

During dry conditions the thermal resistance of the systems is lower than that of traditional systems. The reason for the extra heat transfer is the convection in the air gaps. It is therefore important that the air inlet and outlet are closed under normal conditions to avoid extra heat transfer due to extra convection.

For both roofing systems, the air movement has been investigated as air acts as the carrier of the excessive moisture. The investigations are only performed on a theoretical level as no experimental data exists.

The lack of experimental data is also the reason for the theoretical assessment of the potential drying rate for both systems.

6 Conclusion

Many flat roof constructions run the risk of having the insulation replaced as water is trapped in the insulation between the vapor retarder and the roof membrane resulting in a higher energy loss. The drying of the insulation in such constructions is very difficult and often the only available option is to remove the insulation or add dry insulation on top of the wet layer. To avoid such replacement a series of new roofing systems has been developed for expanded polystyrene, low-density mineral wool and high-density mineral wool. These were developed so repair should be easy and cost-effective in case water enters the insulation. The principle in the developed roofing systems is that air is forced through a gap below the insulation, thereby removing moisture. In the systems based on high-density mineral wool and expanded polystyrene, grooves are created in the insulation, while the system based on low-density mineral wool is using roofing cassettes to establish the air gap.

Another way of improving the roofing system is by decreasing the time from the first water intrusion in the roof construction until the leak is discovered and repaired. This is ensured by using a newly developed moisture sensor which is embedded in the vapor retarder or made in narrow strips which are placed on top of the vapor retarder. By using the moisture sensor the leak in the roof membrane can be pinpointed and sealed and the insulation dried.

Each of the developed systems has been examined with regard to thermal resistance, drying rate, air distribution and economy. These examinations have not yet been supported by field data, but will be so in the near future.

7 References

Danish Building Code (1995) *Danish Building Code*. Ministery of Housing, Copenhagen, Denmark

IEA (1996) *IEA Annex 24 Heat, air and moisture transfer in insulated envelope parts, Final report, volume 1, task 1: Modeling*. International Energy Agency

IEA (1996a) *IEA Annex 24 Heat, air and mositure transport through building materials and components, volume 3, task 3: Material and layer properties*. International Energy Agency

ISO (1997) *Thermal insulation - Building materials and products - Determination of declared and design thermal values*. International Standardization Organization, Geneva. ISO/FDIS 10456:1997(E)

Lund, H. (1985) *Test Reference Years TRY*. Commission of the European Communities, DG XII

Pedersen, C. (1990) *Combined heat and moisture transfer in building constructions*. ph.d.-thesis. Report 214. Thermal Insulation Laboratory. Technical University of Denmark

Rode, C. and Rudbeck, C. (1998) Latent heat flow in light weight roofs and its influence on the thermal performance of buildings. *ASHRAE Transactions* vol 104, pt. 2, pp. 930-940

Tobiasson, W., Korhonen, C., Countermarsh, B., and Greatorex, A. (1983) Can wet insulation be dried out? *Thermal insulation, materials and systems for energy conservations in the 80's*, ASTM STP 789, American Society for Testing and Materials, Philadelphia, PA

Trechsel, H. (1994) *Manual on moisture control in buildings*, ASTM MNL 18, American Society for Testing and Materials, Philadelphia, PA

108 PERFORMANCE OF STRAINED BITUMINOUS WATERPROOFING MEMBRANES UNDER HYDROSTATIC PRESSURE

N. Sahal and E. Ozkan

Istanbul Technical University, Istanbul, Turkey

Abstract

Basement walls and floors of new constructed buildings are sometimes exposed to water that exerts pressure from the outside due to the existence of high level of ground water. Therefore, oxidized and modified bituminous membranes are applied as external or internal tanking for waterproofing. In such applications; the membranes are strained excessively due to the effects of the mechanical agents that are generated from the construction, and meanwhile they are exposed to water pressure that is originated from the groundwater. The watertightness integrity of these membranes are determined according to the existing test methods which can only measure the resistance of unstrained membranes to water pressure. The aim of this paper is to propose a modified test method which evaluates the performance of the strained bituminous membranes under hydrostatic pressure. In this test method; the effects of the mechanical agents on the membranes which are the deformation of the membranes under compressive pressure and the excessive strain of the membranes over the cracks are simulated in laboratory conditions. Then, the conditioned test samples are exposed to hydrostatic pressure with a modified testing equipment and the watertightness integrity of strained test samples under hydrostatic pressure are evaluated with a moisture/water sensor.

Keywords: Bituminous membranes, hydrostatic pressure, modified bitumen, performance standards, testing, waterproofing

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1 Introduction

Bituminous membranes have been used to provide a waterproof barrier in below-grade and roof waterproofing systems for dwellings, industrial and commercial buildings. However, most of the laboratory and field research has only been directed towards gaining knowledge about the agents, and their effects to determine the performance of bituminous membranes used in roofs. Thus, performance requirements in conjunction with test methods have been established for bituminous membranes that are used for roofing. The performance evaluation of the membranes have been determined according to these test methods. However, the same type of membranes are also used in below-grade waterproofing. Obviously, these membranes are exposed to different kinds of agents than the factors defined for roofing, therefore it should be understood that most of the performance test methods can not be used to measure the behaviour of bituminous membranes in-service conditions in below-grade waterproofing.

The bituminous membranes used for below-grade waterproofing systems are oxidized and modified bitumen membranes with polyester reinforcement. Modified bitumen membranes have a widespread usage in below-grade waterproofing systems than oxidized bitumen membranes. However, modified bitumen membranes have been developed to meet the short-coming of the oxidized bitumen to roof climatic factors such as heat, UV radiation, and air; the mechanical performance of reinforcements to meet the requirements for roofing and the final assembly of the roofing system. Although the performance of modified bitumen is better than the oxidized bitumen used for roofing, the performance of both membranes when exposed to below-grade agents are still unknown.

The efficient and effective use of bituminous membranes in below-grade waterproofing systems needs identifying the agents and their effects that should be considered in determining and predicting the performance of the membranes during their service life.

2 The agents and their effects to below-grade waterproofing systems

Basements walls and floors of new constructed buildings are sometimes exposed to water that exerts hydrostatic pressure from the outside due to the existence of high level of groundwater. Water that exerts hydrostatic pressure may damage the structural components such as reinforcing steel corrosion, and make interior spaces inhabitable causing poor indoor air quality. Groundwater may also contain harmful chemicals that may deteriorate below-grade building elements. Thus, groundwater is considered to be the essential agent that naturally occurs externally. Therefore, the fundamental function of a waterproofing system is to prevent the leakage of water into, through or out of the basement floors and walls under hydrostatic pressure without disruption and decay through the whole life of building. Waterproofing is also required to protect the building elements from deterioration when exposed to harmful chemicals that may be found in the groundwater. To perform its function, waterproofing must surround the entire structure with a continuous membrane. To resist the effects of hydrostatic pressure, the membrane, weak in itself, must be loaded by a mass of material at one side and must be protected from damage at the other side. Therefore, the bituminous membranes are sandwiched between two layers in either an 'external tanking' arrangement where the tanking is protected by a protective layer on the excavation side with the structural wall and floor slab internally or alternatively by an 'internal tanking' arrangement where the structural wall and floor slab retains the excavation and sandwiches the membrane with the protective layer internally. In both cases, care must be taken to avoid thinning, puncturing of the membranes and in formation of joints, laps and corners. Besides, the membrane must have good suitability to substrate such as they must not slide on vertical planes and failure must not occur at corners.

In either external or internal tanking, the membrane remains under compressive pressure between the structural floor slab especially under the columns where the loads are relatively high and the concrete blinding. Thus, the ' plastic' bituminous membranes follows the structural deformation.

During construction; after the pumping out of the groundwater is stopped, the rising water will fill the air pockets and push the air to the membrane. The air under pressure will force the membranes which may not be airtight and as a result, air will penetrate through the membranes providing easy access for the water afterwards.

In external tanking, the concrete blinding; and in internal tanking, the structural floor slab, in contact with the membrane, may be attacked by harmful chemicals that are found in the groundwater such as sulphates. The effect is dramatic, as the pressure will force the aggressive chemicals into the concrete through the whole life of building. Sulphates will react with the hardened cement paste of the concrete to form either ettringite or gypsum which eventually will cause the cracking and spalling of the concrete unless preventive measurements are taken.

In external tanking, the surface of the reinforced concrete structural slab in contact with the membrane may also crack due to the excessive tensile stress met by the steel reinforcement. Shrinkage may occur in concrete building elements when concrete losses its construction moisture. The resultant movement is another cause of cracking on the surface of the structural floor slab and reinforced concrete walls. Due to the cracking, a narrow part of the membrane may strain excessively.

Concisely, the mechanical agents that are generated by the construction strain the bituminous membranes excessively. The mechanical properties of the bituminous membranes may withstand the stresses satisfactorily but on the other hand it is not known whether the strained bituminous membranes will still maintain its watertightness function efficiently or lose its watertightness integrity under hydrostatic pressure.

Therefore, a test program which predicts the performance of bituminous membranes under hydrostatic pressure on the basis of the mechanical agents induced in belowgrade systems is determined. These are defined as:

1. In either external or internal tanking; cracks may not occur on the surfaces of the structural walls where the bituminous membranes are applied, therefore, they do not undergo to any strain and only water pressure is exerted on the membranes,
2. In either external or internal tanking; cracks may occur on the surfaces of the structural walls in contact with the bituminous membranes, therefore, the membranes may strain excessively and meanwhile they are exposed to water pressure,
3. Basement floors in contact with bituminous membranes are under compressive pressure especially under the column bases; therefore, they are strained according to the strain of the structural members and meanwhile they are exposed to water pressure,

4. Basement floors in contact with bituminous membranes are strained due to the compressive pressure and cracks, and meanwhile they are exposed to water pressure.

Therefore, the resistances of both strained and unstrained bituminous membranes to the penetration of water under pressure must be determined by a convenient test method.

3 Current test methods for resistance to water pressure

Some of the current standards and general performance documents comprising the test methods for determining the resistance of bituminous membranes to water pressure are given in Table 1.

Table 1: Current standards and documents

Origin	Designation	Date	Title
1. Canada	CSGB 37-GP-56M	1980	Standard for: Membrane, Modified, Bituminous, Prefabricated and Reinforced for Roofing
2. Denmark	UEAtc General Guideline		
3. Europe	UEAtc MOAT 27	1983	General Directive for the Assessment of Roof Waterproofing Systems
4. France	UEAtc General Guideline		
5. Germany	DIN 52123	1985	Testing of Bitumen and Polymer Bitumen Sheeting and Felts
6. Italy	UNI 8202/21	1981	Test Methods for Roof Membranes
7. South Africa	ACTMAP No:1	1991	General Performance Criteria and Requirements for the Assessment of Roof Waterproofing Systems
8. Sweden	UEAtc General Guideline		
9. Turkey	TS 11758	1995	Polymer Bitumen Sheeting for Fusion Welding for Use in Waterproofing Applications
10. USA	ASTM D 5385	1993	Standard Test Method for Hydrostatic Pressure Resistance of Waterproofing Membranes

TS and DIN proposes the same method where unstrained bituminous membranes are exposed to 200 kPa of water pressure for 24 hours. The leakage is observed by naked eye. The UEAT Guidelines follows the same application but differ in water pressure level which is 60 kPa. UNI proposes a similar method where the unstrained bituminous membranes are exposed to 500, 250, 60 and 10 kPa of water pressure for 24, 24, 1 and 1 hours, respectively. The observation method is same as TS and DIN. These test methods are sufficient to measure the resistance of unstrained membranes to water pressure but can not be developed to measure the hydrostatic resistance of strained bituminous membranes. Therefore; these test methods can only provide a guidance for the watertightness quality of bituminous membranes and can be used to compare the hydrostatic resistance of the membranes, but fall short to determine the performance of strained bituminous membranes under hydrostatic pressure.

CSGB, UNI and UEAT Guidelines propose test methods to determine the resistance of water pressure of strained bituminous membranes. The waterthightness integrity of the membranes which have been subjected to either puncture or flexibility test are determined using a 50 mm. of water column. CSGB and UNI conducts the test for 16 and 24 hours respectively but UEAT Guidelines does not propose any time limit. Leakage of water is observed by either naked eye in UNI and UEAT Guidelines or a filter paper which the colored water stains, as proposed in CSGB. These test methods can not be used to determine the hydrostatic resistance of strained membranes as 50 mm. of water column is not adequate to simulate the hydrostatic pressure. Furthermore, the test method can not be developed for higher values of water pressure.

ASTM has also established a test method which measures the hydrostatic resistance of unstrained membranes under controlled laboratory conditions. Unstrained membranes are subjected to water pressure starting from 103 kPa up to 690 kPa by increasing the pressure in 103 kPa steps each hour. Leakage is observed by naked eye. No correlation has been established between the performance of this test method and that in field. But, among the existing test methods, only ASTM test method is considered appropriate to be modified measuring the hydrostatic resistance of strained bituminous membranes.

4 Modified test method for performance testing of bituminous membranes under hydrostatic pressure

ASTM test method has been modified appropriate for simulating both the effects of the mechanical agents on the membranes which are the deformation of the membranes under compressive pressure and the excessive strain of the membranes over the cracks and the exposure of the strained membrane samples to hydrostatic pressure. Besides, a new observation method which determines whether water molecules has penetrated through the membrane during exposure to hydrostatic pressure, has been developed. The advantage of this sensor is that the penetration of water in either moisture or liquid form can be measured.

4.1 Apparatus

4.1.1 Hydrostatic testing equipment includes a chamber, a clamping bracket, a rubber gasket and fasteners to form the completed assembly, Figure 1.

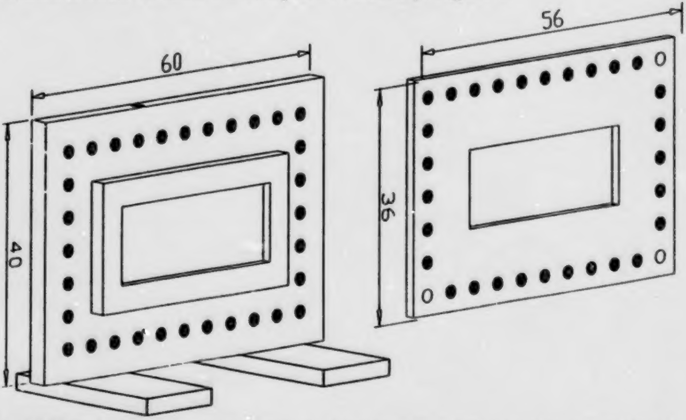

Fig. 1: Hydrostatic testing equipment, the chamber and the clamping bracket

4.1.2 Source of compressed air

4.1.3 Compressive strength testing machine

An universal testing machine is used to load and thus, strain the membrane samples to a predetermined point of compressive pressure and form.

4.1.4 Precast concrete blocks

The precast concrete blocks are used as substrates for the membranes. Two different types of blocks are prepared for the tests. 'Type 1 precast concrete block' is composed of two parts and the width, length and height of one part is 96, 400, and 50 mm., respectively. The dimensions of the concrete block assembly formed by these two parts are 200 by 400 by 50 mm., leaving a gap of 8 mm. between the widths of the two parts. The gap is used to monitor the leakage of water through the membrane during testing. 'Type 2 precast concrete block' has a kerf 4 mm. deep lengthwise down the center of the block and the width, length and height of the block are 200, 400 and 50 mm., respectively. This type of block is used to simulate the cracks that may occur on the surfaces of the substrates. The compressive strength of both types of concrete blocks are determined to be minimum 14.7 N/mm^2.

4.1.5 Metal elements

Metal elements used as tools in testing are steel spacers, steel plates and steel frames. Three types of spacers which differ in thickness are used as tools to simulate the occurrence of cracks on the surfaces of the building elements in contact with the membranes. The width and the length of all the steel spacers are 50 by 50 mm., and the thickness are 6, 8 and 10 mm.

The two steel plates which sandwiches the membrane sample are used as supports to the strained membrane sample after the samples are loaded to a predetermined point of compressive pressure in the universal testing machine. Then, two steel frames are used to fix the membrane sample in strained position. The width, length and height of the steel plates and the steel frames are 200, 400 and 30 mm., and 260, 460 and 2 mm., respectively.

4.1.6 The moisture/water sensor

The sensor utilizes a water absorbent polymer which is a swellable substance that can absorb water molecules and retains it by forming a gel. The thin layer of polymer is deposited between two vertically placed electrode plates which one of them is made of porous metal. The length and the height of the electrodes are 300 and 6 mm., respectively. The two vertical electrode plates with the polymer between is situated in a polycarbonate case which the length, width and height are 300, 6, and 6 mm, respectively. The case is closed on all sides except the front side. The vertically placed porous electrode plate takes place in the front side of the case so that the thin layer of polymer behind the porous plate can directly receive the water molecules which penetrates through the membrane.

The sensor is connected to a 1 M Ω resistor in series. A step-down transformer with an input of 220 V AC and an output of 36 V AC is used and between the terminals of 36 V AC, the serially connected sensor and resistor is placed. The two probes of a high accuracy multimeter are attached to the terminals of the resistor. Because of high impedances and low signal voltages, the measurements are very sensitive to stray noise. To obtain high precision, a Faraday cage is used to isolate and eliminate these noise signals. The transformer, the resistor and the multimeter are placed in the Faraday cage. The cabling of the sensor is shielded. Before making any measurements, the cage is completely closed and latched.

When the water absorbent polymer is dry, its ohmic resistance is very high. Nearly no AC current flows in the circuit; thus, no AC voltage can be measured between the terminals of the resistor. As the water molecules penetrates through the porous electrode plate; the water content of the polymer increases, resulting an decreasement in the ohmic resistance of the polymer. The sensor responds to the change in its environment by increasing its current output, then by running the current through the resistor, an increasing voltage is produced. The multimeter can then monitor the increasing voltage between the terminals of the resistor. The measured data is recorded and stored in a computer program to be evaluated periodically.

The meter readings in terms of AC voltage are related to weight of water molecules in grams by a calibration curve.

4.2 Membrane test samples

Commercially available oxidized and modified bitumen membrane samples that will be used in testing are described in Table 2.

Table 2: Description of membrane test samples

Bitumen membranes	Reinforcement (gr/m^2)	Thickness (mm)	Number of plies
1 APP modified bitumen	Unwoven glass fibre, 60	3	1
2 APP modified bitumen	Polyester, 180	3	1
3 APP modified bitumen	Polyester, 180	4	1
4 APP modified bitumen	Polyester, 180	3+3	2
5 SBS modified bitumen	Glass, 60	3	1
6 SBS modified bitumen	Polyester, 180	3	1
7 SBS modified bitumen	Polyester, 180	4	1
8 SBS modified bitumen	Polyester, 180	3+3	2
9 Oxidized bitumen	Unwoven glass fibre, 60	2,4	1
10 Oxidized bitumen	Woven glass fibre, 200	2,9	1

4.3 Tests for measuring the hydrostatic resistance of unstrained and strained bituminous membranes

Four types of tests related to the test program defined in Section 2, have been established.

4.3.1 Test no 1

This test simulates the condition of the membranes applied on the surfaces of the structural walls where cracks may not occur and measures the hydrostatic resistance of the unstrained membranes. There is no pre-formed strain on the membranes prior to hydrostatic pressure testing. This test will be conducted to all types of membrane samples defined in Section 4.2.

- The procedure

The surface of the Type 1 precast concrete block which will receive the membrane is primed as recommended by the manufacturer. The primer is permitted to dry for the minimum time recommended by the manufacturer. Liquid rubber is then applied to the remaining surfaces of the block. The membrane sample that will be tested is cut from the roll and installed over the primed surface of the prepared block and allowed to cure for the minimum time recommended by the manufacturer. The width and the length of the membrane sample is 200 and 400 mm., respectively. Two pieces of rubber which the width, length and height are 50, 50 and 8 mm., respectively, are placed at the ends of the gap.

Both sides of the rubber gasket are coated with vacuum grease and one side of the rubber gasket is fitted to the test chamber. The width and the length of the rubber gasket is 200 and 400 mm., respectively. The membrane side of the block is set against the gasket and then the steel bracket is set against the back of the block. The fasteners are tightened, clamping the block to the chamber gradually to form the completed assembly. Then the sensor is placed in the gap between the blocks. The vertically placed porous electrode plate faces the surface of the membrane sample. Then the gap is sealed with a strip of rubber in order to prevent the access of humidity to the gap.

With the air vent open, the chamber is filled with water. Then, both the air vent and water valve is closed. The assembly is monitored for leaks for 30 minutes. The air line is attached if no leakage is observed in the assembly. The air pressure is increased up to 50 kPa for one hour, and to 100 kPa for the next hour. The last upward pressure is 250 kPa for 24 hours. Meanwhile the meter readings are recorded in terms of AC voltage and converted to weight of water molecules that passed through the membrane under hydrostatic pressure.

4.3.2 Test no 2

This test simulates the excessive strain of the membranes caused by the cracks that may occur on the surfaces of the structural walls in contact with the membrane and measures the hydrostatic resistance of the strained membranes. The crack widths which will be simulated are 2, 4 or 6 mm. The strain of the membrane caused by the crack width of 2 mm. will be simulated in a membrane sample and the resistance of this membrane to water pressure will be measured. Afterwards, the deformation of the membrane due to the crack width of 4 mm. will be simulated in a new membrane sample of the same type and the resistance of this membrane to water pressure will again be measured. A new membrane sample of the same type will be strained due to the simulation of the crack width of 6 mm. Then, the watertightness level of each strained membrane samples under hydrostatic pressure will be evaluated. This test will be conducted to all types of membrane samples defined in Section 4.2.

- The procedure

The flat surface of the Type 2 precast concrete block which will receive the membrane is primed and after the primer is dried, liquid rubber is applied to the remaining surfaces of the block. The membrane sample that will be tested is cut and installed over the primed surface of the prepared block. The width and the length of the membrane sample is 200 and 400 mm., respectively. With the waterproofed surface down, the block is supported at both ends directly under the kerf. Two steel spacers which are 2 mm. thicker then the width of the kerf are inserted at each end of the kerf. Then the steel spacers are tapped with a hammer to break the balance of the block at the kerf. Thus, the occurrence of 2 mm. wide cracks on the surface of the structural walls which strains the membrane is simulated. In order to obtain the cracks which the widths are 4 and 6 mm., 8 and 10 mm. thick steel spacers are used in new membrane samples of the same type, respectively. The block is installed between the chamber and the steel bracket, and the fasteners are tightened to form the completed assembly. The sensor is then placed in the kerf and the kerf is sealed with the strip rubber. The strained membrane is exposed to the same water pressure levels as described in the procedure of Test no 1, and the meter readings are converted to the weight of water molecules which penetrated through the membrane during the exposure time.

4.3.3 Test no 3

In order to simulate the strain of the membranes when subjected to the compressive pressure under the column bases, a membrane sample will be strained at 10 t of compressive pressure and then, the hydrostatic pressure resistance of this strained membrane will be measured. New membrane samples of the same type will be used for each compressive pressure levels of 20 and 30 t., separately. The strained membranes will be evaluated on the basis of their performances under hydrostatic pressure. This test will be conducted to all types of membrane samples defined in Section 4.2.

- The procedure

The surfaces of Type 1 precast concrete block is prepared as described in the procedure of Test no 1. The membrane sample that will be tested is cut from the roll and placed between the two steel plates. The width and the length of the membrane sample is 260 and 460 mm., respectively. The two steel plates with the membrane sample between is then placed between the two jaws of the universal compressive strength testing machine. The test sample is loaded to a predetermined point of compressive pressure and then fixed between two steel frames while in strained position. The strained test sample with the steel frames is installed to the primed surface of prepared block and the block is installed in the hydrostatic testing equipment. The sensor is then placed in the gap and the gap is sealed. The same water pressure levels are applied to the membrane as described in the procedure of Test no 1. The meter readings are converted to the weight of water molecules by the calibration curve.

4.3.4 Test no 4

This test simulates the strain of the membranes when exposed both the compressive pressure and the cracks and measures the resistance of the strained membranes to water pressure. Initially, a membrane sample will be strained to 10 t. of compressive pressure level. Then, the strained membrane sample will further be strained by simulation of the 2 mm. wide crack and the performance of the strained membrane sample under hydrostatic pressure will be evaluated. A new membrane of the same type will be strained in 10 t. compressive pressure level and this time the crack width of 4 mm. will be simulated on the same sample. Afterwards, the crack width of 6 mm. will be simulated in a new membrane sample of the same type which has already been exposed to 10 t. of compressive pressure. The hydrostatic pressure resistance of the strained membrane samples will be evaluated. The same type of membrane will be strained to 20 t. of compressive pressure level, and the crack width of 2 mm. will be simulated on the same sample. New membrane samples of the same type will be used for each compressive pressure levels of 20 and 30 t., separately. In each compressive pressure levels, a new sample of the same membrane type will be used for the simulation of 2, 4 and 6 mm. wide cracks, separately. All types of membrane samples defined in Section 4.2 will undergo the same procedure.

- The procedure

The Type 2 precast concrete block is prepared as described in the procedure of Test no 2. Then the membrane sample which will be tested is strained in the universal compressive strength testing machine as described in the procedure of Test no 3. The strained test sample with the steel frames is installed to the primed surface of the prepared Type 2 precast concrete block. Steel spacers are inserted into the kerf and tapped with hammer. Thus, the strained membrane is furthermore strained due to the occurrence of crack. The block is placed to the hydrostatic testing equipment. The sensor is then placed to the kerf and sealed with the rubber strip. The predetermined levels of water pressure is applied as described in the procedure of Test no 1 and the weight of water molecules are determined from the calibration curve.

5 Concluding Remarks

Oxidised and modified bitumen membranes are not only used in roofing but below-grade waterproofing as well. The waterthightness integrity of these membranes used in below-grade waterproofing are determined according to the existing waterthightness test methods which do not take into account the mechanical agents that these membranes are exposed to. Therefore, a modified test method which measures the hydrostatic resistance of the membranes when subjected to mechanical agents that occurs in below-grade waterproofing is proposed. This modified test method is considered to be a new approach which will provide a better understanding of the performances of the bituminous membranes used in below-grade waterproofing.

6 References

American Society for Testing and Materials. (1993) *Standard Test Method for Hydrostatic Pressure Resistance of Waterproofing Membranes*. ASTM, ASTM D 5385.

Lufsk, K. (1980) *Yapilarda Su Izolasyonu*, Seyas, Istanbul.

Ozkan, E. (1993) The strength, elongation and toughness of fresh and weathered bituminous roofing membranes. *Building Research and Information*, Vol. 21, pp. 225-234.

109 WATERPROOFING DURABILITY: A SURVEY OF SEVERAL APP MODIFIED ROOF MEMBRANES IN SERVICE FOR MORE THAN 20 YEARS IN EUROPE

P. Sartori

ICITE-CNR, Milan, Italy

Abstract

Eleven APP-modified roofs were randomly selected from a total of 36 for inspection. All of them were installed between 1974 and 1983. Two of them are close to the Alps, two on the sea coast and seven on the plain of river Po. Some samples of membrane were taken from 7 roofs for laboratory investigations about the polymer dispersion grade, cold flexibility, ultimate tensile stress and elongation, joint shear and T peel strength, thickness (if possible). The scope of the work is to verify the conditions of the roof according to the applied maintenance, its influence on the roof and the membrane behaviour, the laboratory test validity on the evaluation of the expected remaining life and to assume the remaining life of the roof according to the applied maintenance, ordinary maintenance, extraordinary maintenance, no maintenance. Comparison between this work on roofs in Italy and similar works (with the same membrane) on roofs in the Netherlands, Belgium, Norway.

Keywords : APP modified bitumen, natural ageing, waterproofing membrane, roofs

1 Scope

The scope of this work is to make an assessment of current conditions, to measure some parameters of the membranes after about 20 years of service and to express with all necessary reservation the possible remaining life of the roofing.

The company Derbit provided a list of 36 roofs of buildings of different types located in Northern Italy and built between 1973 and 1983 among which 11 roofs have been selected in the provinces of Bologna, Forlì, Rimini, Treviso and Belluno in order to perform a careful visual examination of the roofing conditions. Some

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membrane samples were taken from 7 out of the 11 roofs to determine some characteristics of the membrane after a natural protracted ageing.

2 Investigation of the works

2.1 Pavilion 29 of the international exhibition of Bologna, Viale della Fiera 20

Installed in 1978 on an overall surface of 8000 m^2 in an urban area. Cold and humid climate in winter, hot and humid climate in summer (typical continental climate). Two above-ground floor steel structure; roofing slope about 1-2%.
Roof system consisting of:

- Currugated metal sheet
- PUR panels faced with bituminised paper with 4 to 8 cm thickness, mechanically fixed to the sheet;
- Low-thickness, glass-felt-reinforced, drilled and cork-coated bituminous membrane;
- Oxidised bitumen smearing;
- Torched Derbigum SP 4 mm ;
- Al paint.

On the roof there are several air-conditioning systems, turrets and air vents (picture 1). Maintenance is regular, satisfactory and frequent. Al painting is in good conditions and it is remade every three years (the last one in 1996). There are some patches due to mechanical damages caused by works made after the realisation of the waterproofing (picture 8). The roof is accessible from a large steel staircase leaning on one side of the structure. Overlaps are 6 cm wide, even, they show no disjunction or ungluing and no sign of slipping of the joints (picture 7). There are some small backwater points (pictures 1 and 4). Some cracks and alligatoring concerning just the painting layer (pictures 5, 6, 10 and 11). Photographic evidence pictures 1 to 11.

2.2 Shed and offices of the company Copari s.c.r.l. located in Forlì, Via dell'Industria 72

Installed in 1976 on an overall surface of 1300 m^2 in an urban-agricultural area with a moderate continental climate. 1-2 above-ground floor pre-cast concrete structure; 1% slope.
Roof system consisting of:

- Pre-cast concrete roofing-tiles of about 2m x 4m resting on inverted-U beams;
- 3 cm-thick glass wool panel coated with bituminised paper;
- Oxidised bitumen smearing;
- Bituminised feltpaper;
- Torched Derbigum SP 4 mm ;
- Al paint.

On the roof there are several roof lights, turrets and downpipes drains (picture 12). Maintenance is regular, satisfactory and frequent. A1 painting is in fairly good conditions and it is remade every three years. There are some small patches at one end of the beams. The roof is not directly accessible. The breadth of overlaps lies between 8 and 10 cm, with no disjunctions, ungluing or any sign of slipping of the joints. Macrophotography No. 17 shows the painting alligatoring and the evident but intact glass-felt reinforcement while macrophotography No. 19 shows a joint which is not welded on its external side. Photographic evidence pictures 12 to 19.

2.3 Ente Fiera di Forlì (Exhibition Organisation) located in Forlì, Via Punta di Ferro

Installed in 1976 on an overall surface of 13000 m^2 in a rural area with a moderate continental climate. 1-2 above-ground floor steel structure; 1% slope.
Roof system consisting of:

- Corrugated metal sheet;
- 3 cm-thick glass wool panel coated with bituminised paper;
- Oxidised bitumen smearing;
- Bituminised feltpaper;
- Torched Derbigum SP 4 mm ;
- A1 paint.

No maintenance at all: no further maintenance works have been made since the painting applied in 1976 (picture 20). Dirt, rubbles, pieces of glass or remains from successive works on windows, sheet-metal elements, etc. and also presence of musk (pictures 21, 22 and 23). Appearance of the membrane is good, there are no cracks, fissures, alligatoring. Macrophotographies No. 25 and 26 show the glass-felt reinforcement completely exposed and in some points even torn. 8-10 cm lateral overlaps, 10-15 cm end overlaps with no disjunction or ungluing, no sign of slipping of the joints. Photographic evidence pictures 20 to 26.

2.4 Parco tematico sul mare (ex Colonia le Navi) (Sea park) located in Cattolica (RN), Piazza delle Nazioni 1/A

Installed in 1983 on an overall surface of 17000 m^2 on the seaside. 2-4 above-ground floor laid-concrete structures: slopes are variable (vaults are more or less marked).

Roof system consisting of:

- Laid concrete substrate
- Oxidised bitumen smearing;
- Jute canvas;
- Oxidised bitumen smearing;
- Torched Derbigum SP 4 mm ;
- A1 paint.

Poor maintenance, painting is remade only in some points. 2-3 cm lateral overlaps, 5-8 cm end overlaps, with 4 thicknesses and no disjunction or ungluing, no sign of slipping of the joints. Excellent adhesion both in the structural elements and in vertical elements so that it was not possible to take any sample. Some small blisters with 30-50 cm diameter. Many patches and considerable damages due to vandalism (chair feet, shoe-heels, aerials, etc.), see pictures 31 and 32. No ordinary maintenance at all (pictures 34 and 35). Some blisters have been cut in order to check out any presence of humidity or water between waterproofing and substrate: no water seepage.
Photographic evidence pictures 27 to 35.

2.4 Corial-Unifeed in Rimini, Via G. Palmiri

Installed in 1981 on an overall surface of 1000 m^2 in a rural area at about 1 km away from the seaside. 1 above-ground floor pre-cast concrete structure: 3% slope.
Roof system consisting of:

- Pre-cast concrete roofing-tiles;
- Dry-applied bituminised glass-felt;
- 6 to 7 cm-thick glass wool panel coated with bituminised paper;
- Oxidised bitumen smearing (light);
- Torched Derbigum SP 4 mm ;
- Al paint.

Poor maintenance, very old painting, which in some areas has even faded away (pictures 36 and 44). The breadth of overlaps lies between 5 and 8 cm, with no disjunctions, ungluing or any sign of slipping of the joints. The membrane is detached from the substrate in vertical elements (pictures 41, 42 and 43); disjunctions and cracks can be found at the bottom of the turrets. On the roof there are several turrets, some air-conditioning systems and air intakes; there is also a Faraday cage with metal strip resting directly on the membrane (picture 37, 40, 41 and 45). The general appearance of the roof extrados shows an irregular plane, with laps at the edges of the insulating panels (picture 39 and 44). The panels seem to have moved in a meaningful way due to wind effects (???). Few cracks can be noticed on the edges of the omega-shaped structural expansion joints (picture 45 and macrophotographies 46 and 47). It can be noticed that the waterproofing layer has moved from its original position probably due to wind effects (picture 39). Some backwater in points of minimum slope. Photographic evidence pictures 36 to 47.

2.5 Segheria Antonelli (saw-mill) in Budrio di Longiano (FO), Via Massa 2244

Installed in 1981 on an overall surface of 800 m^2 in a rural area with a moderate continental climate. 1 above-ground floor cement-brick structure with vaulted roof.
Roof system consisting of:

- Vaulted cement-brick substrate;
- Torched Derbigum SP 3 mm ;
- Al paint.

Poor maintenance, very old painting, which in some areas has even faded away (pictures 48, 49, 50 and 51). 3-5 cm lateral overlaps, 5-6 cm end overlaps, with 4 thicknesses and no disjunctions, ungluing or any sign of slipping of the joints (picture 49). Excellent adhesion both in the structural elements and in vertical elements. Two small patches. Some small laps in the lower parts of the vaults (pictures 50 and 51). In some points the glass felt reinforcement is exposed (macrophotography 52). Photographic evidence pictures 48 to 53.

2.6 Magazzino ortofrutticolo Lugaresi (fruit and vegetable store) in Gambettola (FO), Via Martiri di Ungheria

Installed in 1979 on an overall surface of 5000 m^2 in a rural area with a moderate continental climate. 1 above-ground floor cement-brick structure with vaulted roof and lower parts with a slope of about 15%.
Roof system consisting of:

- Vaulted cement-brick substrate;
- Torched Derbigum SP 3 mm ;
- Al paint.

It is the reconstruction of an old brick channel tile roof. Good maintenance, painting is in excellent conditions with some signs of alligatoring (picture 59, macrophotographies 60 and 61). 5 cm lateral overlaps, 6 cm end overlaps, with no disjunctions, ungluing or any sign of slipping of the joints (picture 49). Adhesion is excellent in the structural elements, poor in the channel tiles of the vault and about 50% in the vertical elements. There are many small patches (picture 55). There are also some laps in the lower parts of the stiffening beams (picture 56 and 58). Photographic evidence pictures 54 to 61.

2.7 Marangoni pneus (ex Renova Reifen) in Feltre (BL), Viale Industrie 7

Installed in 1974 on an overall surface of about 1500m^2 (on the whole about 15000 m^2 over the following years) in a rural area with a mountain climate. 1 to 3 above-ground floor cement-brick structure with flat roof; 2% slope in the investigated part.
Roof system consisting of:

- Laid-concrete tiles of about 4m x 4m (finishing of the old waterproofing);
- Cold bituminous paste as preparation of the old substrate;
- Torched Derbigum SP 4mm ;
- Al paint.

It is the reconstruction of an old built-up waterproofing protected by means of laid-concrete tiles. Maintenance is good, painting is in excellent conditions (last painting made in 1995) with some cracks, fissures and alligatoring (macrophotographies 69 and 71). 9-10 cm lateral overlaps, 12 cm head overlaps, , with 4 thicknesses, no disjunctions, ungluing or any sign of slipping of the joints.

Adhesion is excellent in the structural elements. There are some small cuts (3-5 cm long) (macrophotography 68) and some cracks in the membrane (macrophotography 70). It is interesting to remark that the atmosphere is extremely aggressive even if the building is located in a rural area, due to the activities carried out in the plant: metal window frames and casings showed a great degree of corrosion preventing any possibility of recovery (picture 67). It should be pointed out that the roof was misused; in fact it was used as a bearing surface for heavy containers without any protection of the membrane (pictures 62, 63 and 64). Photographic evidence pictures 62 to 71.

2.8 Birreria Pedavena (Brewery) in Pedavena (BL)

Installed in 1976 on an overall surface of about 1000m^2 in a rural area with a mountain climate. One above-ground floor cement-brick structure with flat roof; 2% slope.
Roof system consisting of:

- Laid-concrete tiles of about 4m x 4m (finishing of the old waterproofing);
- Cold bituminous paste as preparation of the old substrate;
- Torched Derbigum SP 5mm ;
- Al paint.

It is the reconstruction of an old built-up waterproofing protected by means of laid-concrete tiles. Maintenance is poor, painting is in very bad conditions (pictures 72, 73, 74, 75, 76, 78 and 79). 8-10 cm lateral overlaps, 12 cm head overlaps, with no disjunctions, ungluing or any sign of slipping of the joints (picture 76). Adhesion is excellent both in vertical and horizontal structural elements. On the roof there are several installations realised over the time (pictures 72, 73, 74 and 75). There are few laps at the edges of the concrete tiles and a big patch (1 x 2 m). Picture 79 shows a non-through crack in the membrane about 2 mm deep at the joint between two concrete panels, protection of the old waterproofing (macrophotographies 80 and 81). Photographic evidence pictures 72 to 83.

2.9 Benetton sport system (ex Nordica Confezioni) in Cusignaga-Giavera del Montello (TV), Via S. Rocco 112

Installed in 1974 on an overall surface of 16000 m^2 in a rural area with continental climate. Two basement floor and one above-ground floor cement-brick structure with flat roof ; 2% slope.
Roof system consisting of:

- Concrete laid in sections of about 4mx 20m;
- 2 cm PUR panels coated with bituminised paper;
- Dry-applied glass-felt;
- 3 mm bituminous membrane reinforced with glass-felt;
- Toeched Derbigum SP 4 mm ;
- Al paint.

The 3 mm glass-felt reinforced bituminous membrane and Derbigum SP 4 mm have been installed with crossed sheets. Maintenance is satisfactory, painting was remade in 1994, great amount of dirt accumulated (pictures 87, 88 and 89). 6 cm lateral overlaps with no disjunctions, ungluing or any sign of slipping of the joints. No adhesion of both horizontal and vertical structural elements to the substrate, excellent between membranes. On the roof there are many roof-lights. Photographic evidence pictures 84 to 89.

2.10 Maglificio Fontane (Knitwear Mill) in Fontana di Villorba (TV), Via Fontane 62/A

Installed in 1975 on an overall surface of 4600 m^2 in a rural area with continental climate. One above-ground floor cement-brick structure with flat roof ; 0-1% slope.

Roof system consisting of:

- Concrete roof-tiles;
- 4 cm PSE panels;
- Dry-applied glass-felt ;
- 3 mm bituminous membrane reinforced with glass-felt ;
- Torched Derbigum SP 4 mm ;
- Al paint.

Maintenance is satisfactory, painting was remade in 1996. 6-8 cm lateral overlaps, 10 cm head overlaps, with no disjunctions, ungluing or any sign of slipping of the joints. No adhesion of both horizontal and vertical structural elements to the substrate, excellent between membranes. On the roof there are many roof-lights, turrets and chimneys. Some backwaters, some patches close to one of the aerials and many small laps due to the crushing of PSE causing the exposure of the edges of the insulating panels (pictures 90, 91 and 92). Photographic evidence pictures 90 to 97.

3 Preliminary general considerations

The service life of the investigated roofs lies between 15 and 24 years.

The general appearance of roofs, but for one single case, proves to be satisfactory, without meaningful laps, blisters or bulge, cracks or fissures of the membrane which may compromise its serviceability.

The joints do not show any sign of slipping, opening (but for one point) or loss of tightness.

The glass-felt reinforcement is always evident, even if in a couple of cases it is torn, but without degradation signs to compromise its serviceability. Rips are always close to the joints and they are considered to be due to the jointing procedure.

Except for three cases, all the roofs showed that neither ordinary nor extraordinary maintenance had been carried out as it may be noticed by the very old painting, which was never remade and in some cases had practically vanished, by the presence of rubbles, vegetation, dirt, etc.

No loss of impermeability and no water seepage in the building were recorded in the investigated roofs, including those with no maintenance. Several patches were found on nearly all the roofs, mostly due to following works made on the roofs also a long time after they were built (aerials, air-conditioning systems, chimneys, etc.)

Table 1: Recapitulation of the membranes investigated in ICITE's laboratories

Sample	Intended use	Locality	Year of installation
1	Exhibition Pavilion	Bologna	1978
2	Storehouse	Forlì	1976
3	Exhibition Pavilion	Forlì	1976
5	Food store	Rimini	1981
8	Tyre factory	Belluno	1974
9	Brewery	Belluno	1976
10	Sportswear manuf. and store	Treviso	1974

4 Experimental tests carried out on the samples

We tried to obtain the necessary specimens for the envisaged tests from the samples that were taken on the building sites. This procedure was not very easy since it consisted in separating the Derbigum SP membrane from everything it was firmly connected to, trying not to damage the material in order to avoid invalidating the experimental results. We failed to obtain specimens for a meaningful measure of the thickness because the lower face of the membrane was always partially melted and other materials were sticking to it. Both the amount and the size of the obtained specimens are considered to be sufficiently meaningful for all the other tests.

All the tests were carried out by complying as strictly as possible with the UEAtc Directives on modified APP bitumen membranes.

5 Microscopic analysis

Pictures 98 to 103 illustrate the photographic shots of the specimens obtained from the different samples taken by means of a fluorescent microscope (200x). As it may be observed, in all the investigated specimens but one (picture 98) there is a good dispersion, no phase separation occurs even after the long ageing of the membrane with a strong and prolonged exposure to radiating agents.

Halfway through the '70s, when the investigated membranes were produced, the techniques used to check the dispersion of the compound were rather rudimentary; fluorescent microscope was still unknown; the characteristics of the compound were checked through measures of R.B., penetration at 25°C and Fraass point. Nevertheless the compound showed a good dispersion which was kept over the time. It should be reminded that in those years polymers were very cheap and of great quality because they were by-products of isotactic polypropylene and very rich in ethylene copolymers (about 50% of propylene was polymerised in a useful form –

isotactic – while all the rest remained as atactic polymer and copolymers produced by the regeneration of catalysts); therefore compounds were very rich in polymers with respect to current ones (about 5% more).

6 Cold flexibility temperature

The cold flexibility temperature of all the taken samples resulted to be higher than room temperature (23°C ± 2°C) so it was decided not to carry out any tests .

7 Thickness

The measurement of thicknesses of the specimens obtained from the samples was not deemed to be meaningful for different reasons. First of all, during the installation phase the membrane melts and, as a consequence, its thickness changes; secondly, when the membrane is torched to the underlying layer the two membranes interpenetrate so that it becomes very hard to find the separation line between them; thirdly, the initial thickness was not known precisely enough to come to sound conclusions (at that time membranes were sold by weight and not by thickness – thickness was merely considered as a very general characteristic).

It must be pointed out that for this type of membrane the glass-felt reinforcement is visible on its external face. As a consequence, the glass-felt reinforcement has a protective function on the membrane for what concerns both the erosion phenomena, and therefore the loss of thickness, and the protection against the outdoor atmospheric agents beside ensuring its main function of providing an excellent dimensional stability.

Thickness values measured on the samples (for information).

Sample 1	thickness	about 4.2 mm
Sample 2	thickness	between 4.2 and 4.6 mm
Sample 3	thickness	between 3.6 and 4.0 mm
Sample 5	thickness	cannot be measured
Sample 8	thickness	between 3.81 and 4.02 mm
Sample 9	thickness	between 5.02 and 5.25 mm
Sample 10	thickness	between 3.70 and 4.40 mm

8 Mechanical characteristics under tensile stress

The measurement was carried out on 3 specimens (direction L) and 3 specimens (direction T) taken from each sample, according to the UEAtc Directives (useful length of the specimen: 200 mm; width: 50 mm; tensile speed: 100 mm/min). The breaking load with the relevant elongation are hereafter given as the mean values of 3 samples .

Table 2: The breaking load with the relevant elongation

Sample	Breaking load (N/5cm)		Elongation at break (%)	
	L	T	L	T
1	542	305	2.3	20.6
2	666	541	2.6	2.0
3	548	640	17.3	50.3
5	674	363	2.6	9.2
8	766	471	2.9	2.7
9	823	531	3.2	2.8
10	624	516	14.0	48.3

9 Shear resistance of the joints

The measurement was made only on lateral joints due to the difficulty in making the sampling, according to the prescriptions of the UEAtc Directives (useful length of the specimen: 200 mm; width: 50 mm; tensile speed: 200 mm/min).

Table 3: Shear resistance of the joints

Sample	Breaking load (N/50mm)	Type of failure
1	395 N/50mm	Break out of the joint
2	465 N/50mm	Break out of the joint
3	694 N/50mm	Break out of the joint
5	439 N/50mm	Break out of the joint
8	719 N/50mm	Break out of the joint
9	532 N/50mm	Break out of the joint
10	495 N/50mm	Break in of the joint

10 Peel resistance at 180° of the joints

The T-peel resistance of the joints was measured on the specimens (length ~ 300 mm and width 50 mm) obtained from the lateral joints of the samples taken. Tensile speed is 100 mm/min. The values illustrated below are: mean value of the load measured in the central area of the specimen (between 25% and 75% of the tested length corresponding to a minimum of 150 mm), minimum and maximum mean value within the above mentioned limits.

Table 4: Mean values of the load measured in the central area of the specimen

Sample	Mean load	Max. load	Min. load	Type of failure
1	90.2	146.5	31.0	Delamination of reinforcement 50%
2	101.2	148.4	31.9	Delamination of reinforcement 100%
3	10.7	17.05	7.0	Defective weld
5	37.7	51.9	25.1	Delamination of reinforcement 100%
8	75.8	114.5	35.9	Delamination of reinforcement 100%
9	53.9	97.8	27.0	Delamination of reinforcement 100%
10	25.8	44.1	16.0	Delamination of reinforcement 100%

12 Conclusions

Generally speaking, all the roofs are in good conditions showing a more than acceptable Derbigum SP membrane behaviour for what concerns its serviceability.

From the appearance point of view, it is evident that no maintenance is made both for what concerns the conditions of the painting and the presence of dirt.

Although in the '70s there were no codified rules for installation, which is an aspect that has not yet been provided for by regulations today, installation was carried out in a sufficiently correct way: it may be remarked that even if the installers had a traditional expertise, only one roof was installed with crossed sheets and that in most cases mistakes are to be ascribed rather to a poor design than to installation itself.

Except for the Exhibition Pavilion of Bologna, the insulating panels were never mechanically fixed, but, at most, glued by means of bitumen; for this reason the membrane had to bear great stresses due to the movements of the insulation. Anyway, this did not affect the good behaviour of the waterproofing over the time.

After an average 20-year service life the membranes' characteristics proved to be rather satisfactory, even if they just had, at most, a light aluminium paint-based protection (which, besides, was hardly ever periodically remade).

The mechanical characteristics under tensile stress showed very high resistance values in the longitudinal direction (about 660N/5cm) which underlines the excellent behaviour of the glass-felt reinforcement, even if this is evident on the surface; on the contrary, the values in the transversal direction, where the glass-felt reinforcement has less influence, proved to be clearly lower (about 480 N/5cm) although very good indeed. It should be recalled that these product families have an anisotropic behaviour even when they are new. We failed to trace back the types of reinforcement used at that time: glass-felt had longitudinal reinforcing elements with a 15 to 30 cm variable pitch while the different types of polyester were of very different quality as to their nature, weight in grams and mechanical characteristics. Except for two cases, elongations showed a remarkable decrease. But after a 20-year service life the membrane has reached its balance that, but for cases of structural failure (which would entail more serious consequences for the work), should no more be altered. Shear resistance values of the joints proved to be excellent (about 535N/5cm) with failures always occurring out of the joint; the same applies to T-peel resistance whose values always exceeded the minimum requirements, but for one sample which showed a clear exfoliation between the membranes (defective welding).

Also the adhesion of the membrane to underlying layers proved to be excellent, even if it has not been measured, so that in some cases it was impossible to take samples of a meaningful size without seriously damaging the samples.

To sum up, the investigation carried out allows to state that:

- the maintenance of the protecting painting being poor, the surface glass-felt has acted also as protection of the membrane;
- with a good maintenance of the painting, a good ordinary and regular maintenance of the roof and envisaging some works of extraordinary maintenance, the roofs considered by this investigation may still have a more than 10-year duration ahead;
- the roofs considered by this investigation having been selected at random, there is no reason to think that other roofs realised with Derbigum SP will not behave in the same way relying on a remaining service life of not less than 10 years, provided that the above mentioned ordinary and extraordinary maintenance rules are respected.

In considering the analogous studies made in Europe by other Institutes[123456], it appears very clearly that the conclusions are the same, nevertheless the climatic conditions and the installation techniques could be very different from those utilized in Italy. This supports our point of view that this kind of membranes could have a very long service life yet.

13 Acknowledgements

The author acknowledges the firm PRS-DERBIT of Italy which made this research possible.

14 References

A complete information with data and pictures can be found in:
Ricerca sullo stato di alcune coperture impermeabilizzate con Derbigum SP - ICITE-CNR Technical Report N°2678/RT/98 - Milano I - Sartori et al. 1998

Etude sur le comportement des toitures Derbigum dans le temps - Rapport N°90-B-0553 - BDA - Gorinchem ND - 900312

Duurzaamheid van Derbigum-daken in de praktijk- Report N°95-B-0513 - BDA - Gorinchem ND - 950725

Investigation of Derbigum SP used as single-layer roofing membrane - N°03651A - Norges Byggforskningsinstitutt - Oslo N - 930329

Field investigation of Derbigum SP used as single layer bituminous roofing membrane on Svalbard - N°08178 - Norges Byggforskningsinstitutt - Oslo N - 961101

Examination of roofs covered with Derbigum - I. Samuelson - Sveriges Provnings-Och Forskningsinstitut - 941212

Comportement de toitures plates avec membranes Derbigum - N°DE 78A623/LEF 2944 - CSTC - Bruxelles B - Nov 1996

110 NATURAL AGEING OF WATERPROOFING MEMBRANES

P. Sartori, M. Becuzzi, and C. Pollastro
Consiglio Nazionale delle Ricerche, Milan, Italy

Abstract

Twelve roofing systems (of about 300m^2 each) have been installed by using four different types of membranes (two with plastomeric modified bitumen and two with PVC) and five types of thermal insulating materials. The systems consist of two inverted roofs, seven insulated roofs with heavy protection, two insulated roofs with factory-applied surfacing in mineral granules and one with protection inside the coating medium (self-protected). The systems were constructed during 1990 and, after a period of three and six years, samples of membranes were taken to assess the extent of natural ageing . We report the results compared with the original values and with the results obtained after the artificial ageing following the UEAtc procedures. Each system has been monitored by means of thermistors placed at different levels into the roofing systems to measure and record continuously the temperatures achieved by the systems layers. We discuss the thermal behaviour of the various systems and how the heavy protection can or cannot help the membrane to support the ageing stresses. A preliminary survey on this subject was presented at 2nd Congreso Internacional en Argentina de Tecnologia de Techos which was held in Buenos Ayres in 1994.

Keywords: APP modified bitumen, natural ageing, PVC, waterproofing membrane,roofs

1 Introduction

The aim of this work is to try to increase the knowledge about the possibility of predicting the behaviour of waterproofing membranes in service. To this end, some characteristics of products aged in service were compared both with those of the same non-aged products and with those resulting from artificial ageing. The evaluation of

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behaviour over the time of waterproofing membranes making up a roofing system is, to date, one of the problems still awaiting a satisfactory solution. There are several methodologies of artificial ageing able to verify the degradation of membranes over an acceptable period of time, but results are not fully satisfactory since the parameters measured after artificial ageing did not conform to the same parameters measured after natural ageing. Everybody is aware of the extremely important role played by design and most of all by the installation of waterproofing membranes on the behaviour of the waterproofing system in service. Duration over the time of a waterproofing work certainly depends on the characteristics of the used products, but it depends even more on the installation quality. It must be reminded that:

- the effect of the coupling between thermal insulations and membranes is not yet completely known, mainly due to the interaction among components of different chemical nature and to actual stresses (mechanical, thermal, radiating, etc.) membranes are exposed to;
- in choosing artificial ageing methods, it is advisable to consider the influence of chemical components and therefore of their mutual action;
- artificial ageing methods existing today can not be differentiated due to the above consideration, since they must guarantee a comparison between materials with the same final intended use.

Until 1990, only few scientific reports and papers had been published on this subject: a study on the temperature reached by waterproofing systems[1], a work concerning the behaviour of some SBS systems[2], other studies concerning particular aspects related to natural ageing (blistering, slippage, opening of joints, alligatoring, etc.)[3,4,5,6]. Over the last few years instead, many papers were published concerning the ageing of waterproofing systems; a careful look at the proceedings of all the symposia dealing with waterproofing subjects will be enough. According to what is stated above, the paper attempted to study the behaviour of different waterproofing systems in service and to compare the results obtained in service with traditional laboratory results.

The construction of CNR Science Park in Roma-Tor Vergata made available about $2000m^2$ of roofings, which gave the opportunity of studying waterproofing systems in real working conditions. The construction of the building started in 1989; at that time, it was foreseen that the building would be completed and occupied by 1992. Due to some economical, red-tape and several other problems, construction ended in 1994 but the Science Park is going into operation only now. We were therefore forced to modify our original programme and many of our goals are still to be achieved.

2 Work programme

The study was based on the construction of 12 waterproofing systems fitted with many sensors for the continuous recording of temperatures in different points and on different levels of the waterproofing systems. Some samples of the systems were expected to be taken at fixed intervals to measure some propierties to be compared with similar values measured on new and artificially aged products. Due to the above mentioned delays, the first samples were taken three years after installation, but it was not possible to perform the continuous temperature recording, both because the building was not provided with external walls until 1994, and because of vandalism (removal or destruction of temperature sensors and recording systems) linked to the long construction delays and not occupancy of the building . Only the last sampling, carried out in 1997, met the initial requirements even if the buildings were not actually occupied and, as a consequence, indoor environmental conditions were not controlled by heating and cooling systems. The buildings for which the roof systems were studied, consist of three-storey above ground steel structures with concrete floor cast on a corrugated metal sheet. A layer of LECA lightweight concretewas used to provide slope. The available surface was subdivided into twelve sections, by making use of valleys and ridges. A different waterproofing system has been realized on each of these sections, by using the most commonly employed materials in Italy. For architectural reasons, it was necessary make the external surface as similar as possible to the other roof systems of the Science Park (that were not includedin this experimentation).

The examination concerned the following elements:

- 2 waterproofing systems:
  inverted roof
  warm roof
- 2 membranes families:
  reinforced PVC
    a) normal, 1,5 mm, without backing
    b) suitable for pedestrian access and glued, 4.5 mm, with non-woven backing
  APP modified bitumen
    a) without surface protection
    b) with mineral surface protection
- 5 insulation boards:
  ship-lap extruded PSE, 40 mm
  PUR, 40 mm, with both faces covered with bituminized kraft paper
  cellular glass, 50 mm
  ship-lap expanded perlite boards, 40 mm
  glass-wool boards, 40 mm, with one face coated with oxidized bitumen reinforced with glass-felt,

put together to make up the twelve systems.

3 Materials

3.1 Waterproofing membranes

PVC: both membranes had the same polymer and the same reinforcement (glass-felt without longitudinal bracings, 50 g/m^2); it is therefore logical to presume that the membranes would have the same behaviour after ageing

APP MODIFIED BITUMEN: both membranes had the same coating medium and the same reinforcement (170 g/m^2, non-woven Polyester spunbound); the only difference was the factory-applied surfacing with slate flakes (1000 g/m^2 ± 250 g/m^2) .

All the products were certified by an ICITE technical agrément.

3.2 Vapour barrier layer

A 3 mm APP modified bitumen membrane was used in systems A2, C1, C2, C3 and C4. A 0.35 mm LDPE film was used in systems B2, B3 and B4.

3.3 Continuity and/or vapour barrier layer

A 3 mm APP modified bitumen membrane was used in systems AB, C1, C2, C3 and C4. *(Continuity layer see Appendix)*

3.4 Separation and/or slip layer

According to the circumstances, the material employed was:
200 g/m^2 non-woven polyester
0.35 mm or 0.15 mm individual, doubled or combined LDPE films.

3.5 Insulation component

boards (120 x 60 cm) in case of PSE, PUR and expanded perlite
boards (60 x 45 cm) in case of cellular glass
boards (120 x 100 cm) in case of glass wool

4 Sampling

The first sampling was carried out in 1993 by taking from each system a sample of the membrane alone (about 1 x 1 m) including one longitudinal seam. At the beginning of 1997, another sampling was carried out according to the same criteria used for the first one. During this second sampling, no sample of system BC could be taken since the whole surface was covered with several types of equipment . During both samplings, the temperature monitoring system had to be restored.

5 Experimental results

All experimental procedures were carried out according to the provisions of UEAtc Technical Guides for the issue of technical agrements for roofings (formerly Common Directives)[7].

None of the tested systems showed leaks after 6 years. Specimens of all the membranes, subjected to the watertightness test according to UEAtc (60 Kpa pressure) did not show any leakage.

After 6 years, the visible surface of sysem A2 (PVC accessible to pedestrians) appeared to be in good condition, with no observable defects, while those of systems C3 and C4 (APP modified bitumen, factory-surfaced with mineral granules), although free of visual defects such as alligatoring, wrinkling, ridging or shrinkage, yet showed a quite meaningful loss of mineral granules, but proved to be satisfactorily protected. In the remaining inverted roof and warm roof systems, with membranes under a heavy protection (concrete slabs on plastic supports), the membrane surfaces did not show any defect, apart from the supports' imprint. In system B1, consisting of cellular glass laid with hot bitumen, separation layer in 200 g/m^2 non-woven Polyester, 1.5 mm PVC waterproofing membrane, LDPE slip layer, separation layer in 200 g/m^2 non-woven

Polyester, slabs on supports, the PVC coat was found to be considerably spotted with brown oils diffusing from the bitumen.

5.1 PVC membranes

The examination of data contained in Tables 1 to 4 did not outline any great decrease of the measured characteristics after a 6-year natural exposure, except for the values of elongation at break which showed a decrease both in L and T direction of about 10% after artificial ageing and 20% after natural ageing both after 3 and 6 years. As regards the joints' shear strength, it is difficult to come to any conclusion since joints prepared in the laboratory are always better than the ones constructed on site. It is hard to make the same remarks concerning the PVC membrane accessible to pedestrians, since the results obtained are uncertain over the different tests due to the poor amount of material which did not allow for a deeper analysis. Therefore, concerning PVC membranes, it seems that the artificial ageing methods, used following UEAtc Guideline, fairly correspond to a natural ageing at least over a 6-year period.

Table 1 : PVC membrane 1.5mm reinforced with a glass felt of 50g/m^2
(* 1-month ageing)

Characteristics months at 80°C		Nominal	Values on new values	Values on samples samples after 6
Thickness (mm)		1.5±5%	1.445	not measured
Mass per unit area (Kg/m^2)		1.9±5%	1.82	not measured
Tensile strength at break	L	•600	693	679
(N/5cm)	T	•600	657	698
Elongation at break	L	•200	301	271
(%)	T	•200	286	259
Plasticizer content (%)		32±2	30,9	28,9
DHC (min)		not declared	211	74
Cold flexibility (°C)		•-20	-35	-35
Joint resistance in lap shear (N/5cm)		not declared	627	622*

Table 2 : PVC membrane 1.5mm characteristics after a 3-years natural ageing

Characteristics		System A1	System B1	System B2	System B3	System B4
Tensile strength at	L	708	675	694	698	706
break (N/5cm)	T	644	622	641	643	663
Elongation at break	L	250	249	252	244	247
(%)	T	216	231	222	223	235
Cold flexibility (°C)		-30	-30	-30	-30	-30
Joint resistance in lap shear (N/5cm)		573	607	624	581	549

Table 3 : PVC membrane 1.5mm characteristics after a 6-years natural ageing

Characteristics		System A1	System B1	System B2	System B3	System B4
Tensile strength at	L	671	637	667	653	667
break (N/5cm)	T	625	585	621	623	639
Elongation at break	L	249	253	251	240	244
(%)	T	223	221	224	228	228
Cold flexibility (°C)		-30	-30	-30	-30	-30
Joint resistance in lap shear (N/5cm)		581	547	577	443	622

Table 4 : PVC membrane 4.5mm reinforced with a glass felt of 50g/m^2 and with backing

Characteristics		Nominal	Values on values	Values on samples aged new samples	
				3 years	6years
Thickness (mm)		4.5±5%	4.45	not measured	
Mass per unit area (Kg/m^2)		3.4±5%	3.42	not measured	
Tensile strength at break	L	•750	829	888	829
(N/5cm)	T	•750	620	862	902
Elongation at break	L	•80	61	86	112
(%)	T	•80	120	106	91
Plasticizer content (%)		32±2	30.8	not measured	
DHC (min)		not declared	211	not measured	
Cold flexibility (°C)		•-20	-35	-30	-30
Joint resistance in lap shear (N/5cm)		not declared	900	884	895

5.2 APP modified bitumen membranes

The same comments can not be made for APP modified bitumen membranes. Bituminous membranes without surfacing: the examination of data contained in the tables 5 to 7 shows that breaking loads decrease only lightly due to exposure; in fact, the values of both new and aged products always fall within the ± 20% tolerances with respect to nominal values, while some decrease may be observed in elongation at break.

Table 5 : APP modified bitumen membrane without factory-applied surfacing

(*1-month ageing at 80°C)

Characteristics		Nominal	Values on new values	Values on samples samples after 6
months at 80°C				
Thickness (mm)		4±5%	4.05	not measured
Mass per unit area (Kg/m^2)		4±5%	4.055	not measured
Tensile strength at break	L	750±20%	785	not measured
(N/5cm)	T	650±20%	730	not measured
Elongation at break	L	50±15	44	not measured
(%)	T	50±15	42	not measured
Heat resistance (°C)		•120	150	120
Cold flexibility (°C)		•-15	-15	-5
Joint resistance in lap-shear (N/5cm)		not declared	735	800*

Table 6 : APP modified bitumen membranes without factory-applied surfacing after a 3-years natural ageing

Characteristics		System AB	System BC	System C1	System C2
Tensile strength at break	L	763	730	627	657
(N/5cm)	T	616	667	587	617
Elongation at break	L	40	33	31	33
(%)	T	40	43	35	38
Heat resistance (°C)		150	150	150	150
Cold flexibility (°C)		0	0	0/-5	0/-5
Joint resistance in lap shear (N/5cm)		608	611	612	595

Table 7 : APP modified bitumen membranes without factory-applied surfacing after a 6-years natural ageing

Characteristics		System AB	System BC	System C1	System C2
Tensile strength at break	L	784	---	679	669
(N/5cm)	T	679	---	605	637
Elongation at break	L	35	---	32	34
(%)	T	34	---	34	35
Heat resistance (°C)		150	---	150	150
Cold flexibility (°C)		>+5	---	>+5	>+5
Joint resistance in lap shear (N/5cm)		609	---	559	572

Anyway, these values still fall within the tolerances required by UEAtc Directives. Likewise, a 20% decrease in lap shear strength may be experienced. Yet, also these values are above the minimum acceptability threshold required by UEAtc Directives, that is 500 N/5 cm. The visual examination of the roofings did not detect any slippage of end joints. No decrease in heat resistance was noticed, which is quite usual with APP modified bitumen products. On the contrary, things seem to be critical when it comes to examining cold flexibility values. The threshold fixed by UEAtc Directives is • 0°C. Values obtained after a 3-year natural ageing on products under heavy protection with slabs on supports, are still acceptable but, according to UEAtc rules, they are no longer acceptable after 6 years. This may lead to state tout court that after 6 years the examined products are at the end of their working life. Yet, waterproofings prove to have an acceptable behaviour, they do not show any defect, there are no losses of tightness and the visual examination does not point out any sign of degradation possibly requiring ordinary or extraordinary maintenance interventions.

Factory surfaced bituminous membranes: what has been stated above concerning membranes without surfacing applies also in this case. It must be added that factory-applied surfacing with slate granules performed very well, protecting membranes the same way heavy protection with slabs does, in spite of the noted slate loss. Following what was stated above, it is possible to consider the factory-applied surfacing with slate granules to be as effective as the heavy protection. In examining the diagrams of temperatures, it is possible to notice the thermal stresses affecting membranes in both cases (tables 8 and 9).

Table 8 : APP modified bitumen membrane with factory-applied surfacing

(*1-month ageing at 80°C)

Characteristics		Nominal values	Values on new samples	Values on samples after 6 months at 80°C
Thickness (mm)		4±5%	4.17	not measured
Mass per unit area (Kg/m^2)		5.1±5%	5.083	not measured
Tensile strength at break	L	750±20%	823	not measured
(N/5cm)	T	650±20%	775	not measured
Elongation at break	L	50±15	47	not measured
(%)	T	50±15	45	not measured
Heat resistance (°C)		•120	150	120
Cold flexibility (°C)		•-10	-15	-5
Joint resistance in lap-shear (N/5cm)			not declared	800 818*

Table 9 : APP modified bitumen membranes with factory-applied surfacing after a 3-years natural ageing

Characteristics		System C3		System C4	
		after 3 years	after 6 years	after 3 years	after 6 years
Tensile strength at break	L	883	888	910	827
(N/5cm)	T	606	606	650	607
Elongation at break	L	38	30	33	31
(%)	T	35	37	40	35
Heat resistance (°C)		150	150	150	150
Cold flexibility (°C)		0	>+5	0	>+5
Joint resistance in lap shear (N/5cm)		607	605	584	587

5.3 Thermal measurements

The acquisition system used for the experimentation consists of a central unit for data acquisition and processing, two peripheral satellite-units connected to the measurement sensors, and a data recording unit. The system uses specific software. It may be programmed directly on the building by means of a personal computer. The sensors used to measure temperature values are thermistors of the NTC Fenwal type. They have a measuring range lying between - 20 and +180°C, a ± 0.2 accuracy and a ± 0.1°C tolerance.

At the beginning, every system under examination was monitored by means of 4 measuring probes in three different points (on the whole twelve probes). Temperatures were measured every twenty minutes. Afterwards, due to vandalism, it was decided to use only one group of probes for each system and temperatures were measured every two hours. The original programme planned for recording temperatures over a two-year period. Due to the above mentioned acts, to the theft of instruments, to tamperings of the system (it must be considered that the measurements were carried out in a working building site), we were able to gather data for only about 180 days.

Three of them were chosen for a closer examination: the coldest (April 7th, 1994 - air temperature between + 3 and + 17°C), the hottest (July 18th, 1996 - air temperature between + 20 and + 36°C), and an intermediate day (May 23rd, 1996 - air temperature between + 11 and + 25°C). Tables 10, 11 and 12 report the numerical values of temperatures in the various layers, while tables 13, 14 and 15, (in appendix) report the temperature patterns of the chosen days. Temperatures in that area must certainly have reached even lower values, but data were no longer available.

5.3.1 Inverted roof (systems A1 and BC)

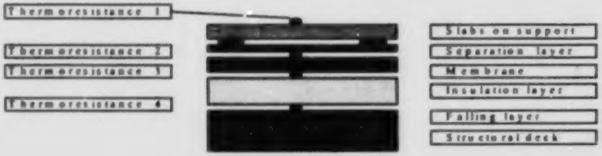
The membrane's temperature is between + 14 and + 34°C with air temperatures of + 3 and + 36°C respectively. As it might be expected, by placing the insulation over the membrane, the inverted roof system can keep the range of temperatures the membrane is exposed to, within reasonable limits, that is to say about 20°C. Heavy protection (slabs on supports) creates a ventilation layer which contributes to decrease temperature, by therefore increasing thermal insulation. It must be reminded that a great part of system BC was covered with technical instruments which were not supposed to be there at the beginning.

5.3.2 Traditional roof

With heavy protection (Systems AB, B1, B2, B3, B4, C1 and C2). The membrane's temperature is between - 1 and + 52°C with air temperatures of + 3 and + 36°C respectively. In these systems, the range of temperatures the membrane is exposed to, becomes quite wide, about 50°C. Also in these systems (slabs on supports) the heavy protection creates a ventilation layer which contributes to decrease temperature, by therefore increasing thermal insulation.

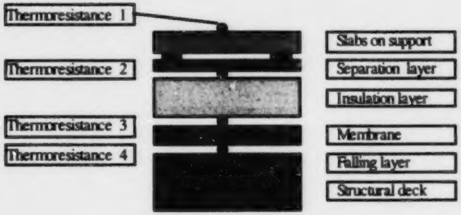
Without heavy protection (System A2 - visible membrane; System C3 and C4 - slate self-protected membrane)

Table 10: The minimum and maximum temperatures of air, surface, membrane, insulation and structural deck measured on three different days


Thermoresistance 1
Thermoresistance 2
Thermoresistance 3
Thermoresistance 4
Slabs on support
Separation layer
Membrane
Insulation layer
Falling layer
Structural deck

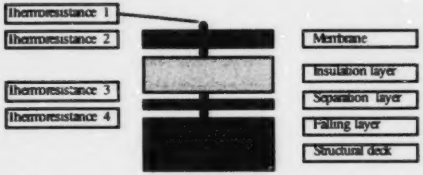
	System			System			System			System			System			System			System		
	apr-	may-	jul-	apr-	may-	jul-	apr-	may-	jul-	apr-	may-	jul-	apr-	may-	jul-	apr-	may-	jul-	apr-	may-	jul-
Air	3	11	20	3	11	20	3	11	20	3	11	20	3	11	20	3	11	20	3	11	20
1	-1	8	17	-3	6	16	-2	6	16	-3	8	18	-1	8	17	-2	7	17	-2	6	17
2	1	9	19	0	8	18	1	11	21	0	7	17	1	9	20	0	9	20	-1	8	18
3	2	10	20	1	8	19	3	12	23	1	8	18	1	10	20	2	10	21	1	9	20
4	12	19	27	11	19	27	9	17	26	14	21	28	15	21	28	12	21	31	15	24	32

Table 11: The minimum and maximum temperatures of air, insulantion, membrane and structural deck measured three particular days in two inverted


Thermoresistance 1
Thermoresistance 2
Thermoresistance 3
Thermoresistance 4
Slabs on support
Separation layer
Insulation layer
Membrane
Falling layer
Structural deck

	System A1			System BC		
	apr-94	may-96	jul-96	apr-94	may-96	jul-96
Air temp	3 17	11 25	20 36	3 17	11 25	20 36
1	-3 35	7 50	17 60	-1 38	8 55	17 61
2	0 28	8 39	18 55	1 34	9 48	18 53
3	14 17	22 26	31 34	14 17	21 24	28 31
4	15 17	22 25	30 33	14 16	21 24	28 31

Table 12: The minimum and maximum temperatures of air, surface, insulantion and structural deck measured on three particulardays in three systems with exposed membranes


Thermoresistance 1
Thermoresistance 2
Thermoresistance 3
Thermoresistance 4
Membrane
Insulation layer
Separation layer
Falling layer
Structural deck

	System A2			System C3			System C4		
	apr-94	may-96	jul-96	apr-94	may-96	jul-96	apr-94	may-96	jul-96
Air temp	3 17	11 25	20 36	3 17	11 25	20 36	3 17	11 25	20 36
1	-4 49	7 50	17 60	-3 48	7 60	16 56	0 39	6 59	17 64
2	-6 51	3 61	15 69	-1 46	8 58	17 68	-3 51	4 67	16 71
3	14 23	22 33	30 39	13 23	23 34	32 41	14 24	23 34	31 41
4	14 22	23 32	30 38	13 23	23 34	32 41	14 25	23 35	31 42

The membrane's temperature is between - 6 and + 71°C with air temperatures of + 3 and + 36°C respectively. The range of temperatures the membrane is exposed to, comes up to nearly 80°C. In all of the systems the support's temperatures lie between + 9 and + 15°C in the coldest moment, while during the hottest hours they lie between + 31 and + 42°C, irrespective of the insulating material or system used.

It must be remembered that in the building no internal temperature controlling system was in operation, that the building was empty, that in the inside a greenhouse effect was taking place and that the thermal insulation was intentionally provided by insulating boards having different thermal conductivity, but the same thickness.

6 Conclusions

The data reported above lead to some final remarks. As foreseen, the thermal stress endured by the waterproofing membranes is a function of the chosen system. As a consequence, the determined values of physical-mechanical properties analyzed as usual (UEAtc Directives) should turn out to be rather different according to the systems. Therefore, the ageing of membranes in inverted roof systems, should be much less than the one of membranes without heavy protection. On the contrary, the experimental results obtained on membrane samples taken after 3 and 6 years in service do not show any meaningful difference. The laboratory analysis of a membrane sample taken from one of the examined roofings, without any information concerning the roofings conditions, may lead to state that APP modified bitumen membranes achieved, or even exceeded, the working life period that could be foreseen according to UEAtc recommendations. As a matter of fact, that's not true, since the examined roofings are still perfectly working and for the time being they are not showing any sign of failure, neither any symptom that could possibly lead to foresee over a reasonable period of time, a meaningful decay eventually requiring a more or less drastic intervention.

In the case of PVC membranes, the data obtained in the laboratory confirm the forecasts of a natural ageing comparable with the accelerated ageing according to UEAtc, apart from system B1 where it may be noticed that the effect of bitumen's most volatile components on the membrane begins to be evident.

In contrast with what stated by M. Puterman[8], APP modified bitumen membranes did not lose their waterproofing characteristics, even if their cold flexibility temperature increased to more than + 5°C.

The results once again confirm and emphasize the opinion that the problems concerning waterproofings are not usually linked to membranes but predominantly depend on installation and design. To obtain final answers about the behaviour of materials, it is undoubtedly necessary to carry out a similar study on a larger number of membranes to be submitted to more extensive examinations by performing physical-mechanical and chemical tests, other than those provided for by UEAtc or other standards.

7 Acknowledgments

The authors acknowledge with thanks Italo Meroni (ICITE) for the contribution in the thermal measurements and the manufacturers of the used products for the, sometimes confidential, information.

8 References

Sartori et al. (1994) Correlation between natural and artificial ageing of waterproofing membranes. *2th Congreso International en Argentina de tecnologia de Techos 1994*,Sesion 7 pp 1-34

Conception et dimensionnement des étanchéites sur isolant-Iournèe d'ètude ITBTP-Annales de l'Institut Technique du Batiment et des Travaux publics-404, may 1982

Duchesne C. (1991) Durability of SBS modified bituminous double-layer system: correlation between performances after artificial and natural aging. *3th International Symposium on Roofing Technology, Montreal 1991*, pp.222-226

ISRR - 2nd International Symposium on Roofs and Roofing, (1981) Brighton

2nd International Symposium on Roofing Technology, (1985) Washington

IWA – 7th International Congress AIE, (1989) Munich

NRCA/NIST – 10th Conference on Roofing Technology, (993) Gaithersburg

UEAtc Technical Guidelines on Waterproofing (the General and Particular ones)

4th International Symposium on Roofing Technology, (1997) Gaithersburg

Appendix

Continuity layer : layer which ensure the horizontal continuity of the support to avoid the effect due to the insulating panel edges. This layer guarantees a continuous surface on a discontinuous layer (or support).

PART SEVEN

ROOFING MATERIALS, COMPONENTS AND ROOF MAINTENANCE MANAGEMENT

ROOF MAINTENANCE MANAGEMENT

111 IMPROVEMENT OPPORTUNITIES IN ROOFING ASSETS MANAGEMENT: EXPERIENCE FROM THE US ARMY

D.M. Bailey and I. Adiguzel

U.S. Army Construction Engineering Research Laboratories, Champaign, IL, USA

Abstract

The Army maintains approximately 170,000 buildings at installations located all over the world. The roofing inventory that protects these buildings comprises a variety of materials and systems. The funds spent annually by the Army to repair and replace roofs make up thirteen percent of its overall building maintenance and repair (M&R) budget. This paper discusses current Army business practices for roof management, investigates technology needs and recommends solutions for bridging existing gaps through proposed process changes and research efforts.

Keywords: roofing systems, asset management, service life performance.

1 Introduction

Roof performance continues to pose a costly and frustrating challenge to the Army. As available M&R dollars and facilities management staffing continue to shrink at Army installations, the concern for roof performance continues to grow. Thus, it is critical for the Army to assess its roofing needs and identify the means for meeting them. This paper defines the Army's roofing assets and performance, and discusses its roof management practices, shortfalls, and needs. Opportunities for improvement through changes in business practices and adoption of technology solutions are presented.

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2 Army roofing assets

2.1 Roofing inventory

The Army is responsible for operating and maintaining approximately 170,000 buildings, which are located at installations located all over the world. These buildings house approximately 1 billion square feet (sf) of floor space (Headquarters, Department of the Army, 1996). The Army has a total of 627 million (M) sf of roofing that protects both mission-specific and family housing buildings. The roofing on mission-specific facilities (training, maintenance, production, supply, storage, troop barracks, medical, administrative, utility) account for 439M sf. Roofing on family housing units account for the remainder.

Army roofing comprises of a variety of materials and systems. These include low-slope roofing systems such as bituminous built-up roofing (BUR) membranes, single-ply sheet membranes, modified bitumen membranes, sprayed polyurethane foams, and metal systems; and steep roofing systems that use materials such as asphalt shingle, metal, clay and concrete tile, slate, and wood. For the 466 million sf of roofing on both mission-specific and family housing buildings at continental US (CONUS) installations only, membrane systems comprise 44% of the total area. Metal and asphalt shingle roofing account for 15% and 35%, respectively (Figure 1). The majority of the membrane roofing is BUR with the remainder being primarily single-ply and modified bitumen systems. The family housing buildings on CONUS installations have predominantly steep roofing, with the majority being asphalt shingles.

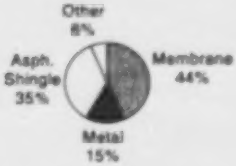

Fig. 1: Percentages of roofing area by type on Army CONUS installations

Based on unit costs (RS Means, 1996), the replacement value for all roofing on CONUS installations is approximately $1.2 billion. Extrapolated for the entire Army, this amounts to more than $1.5 billion in roofing assets.

2.2 Roofing performance

The roofing industry has for many years accepted the notion that BUR systems should be expected to last 20 years. Perhaps for competitive reasons, similar expected service lives for the other membrane roofing systems have been claimed. Based on a 1996 survey of NRCA roofing contractors located all across the United States (Cash, 1997), the actual service life for membrane roofing averages 16 years, only 80% of the industry-standard 20-year design life.

For the purpose of evaluating the performance of Army roofing, the United States Army Construction Engineering Research Laboratory (USACERL) acquired and analyzed ROOFER (Bailey et al, 1989) databases from 21 Army installations (Bailey, 1997). These databases contained inventory data that included historical, design, and construction information as well as condition assessment data for 3059 roof sections totaling 18 million sf of BUR and single-ply membrane roofing. The overall condition of each roof was characterized by the roof condition index (RCI) having a scale of 0 to 100, with 100 being excellent and 33 indicating failure. RCI frequency diagrams for the roofs grouped by age are shown in Fig. 2.

From examining the figures, clearly age is an important factor in overall condition. For the purpose of this study, a roof section having an RCI of 10 points or more below that of a "normal" roof can be considered to be performing unsatisfactorily. Using the data presented in Fig. 2, roofs 5 years or less in age having ratings below "excellent," roofs between 6 and 10 years of age having ratings below "very good," and roofs between 11 and 15 years of age having ratings below "good" were classified as performing at a level lower than should be expected of a typical roof. Based on these conservative criteria, 39 percent of the roofs were performing below their expected level.

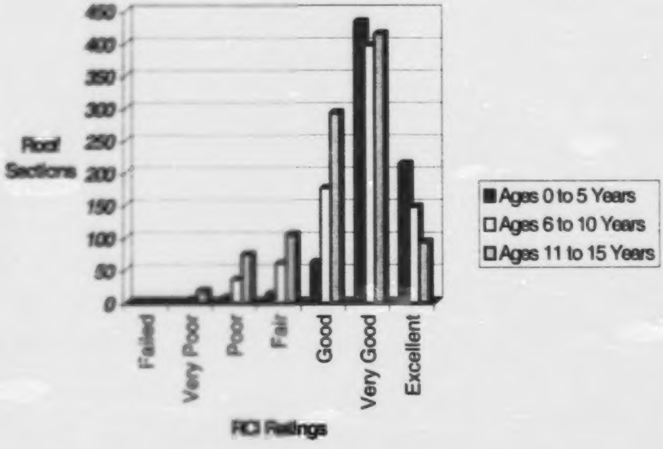

Fig. 2: RCI frequency histograms

Similar indications of Army roofing performance are provided by roof replacement project data compiled from several Army installations that have implemented ROOFER. From a database of 413 roof sections, 70% of the roofs were replaced before reaching their 20-year design life, and almost half were replaced before the age of 16 years. Of the roofs replaced before the age of 16 years, which can be characterized as having below "expected performance," the average age at time of replacement was 12 years.

2.3 Roofing costs

Army roofing M&R costs are not separately tracked but embedded in overall building M&R expenditures. Therefore it is necessary to estimate the roofing portion of the Army's M&R based on private sector experiences. To calculate annual replacement requirements, average durabilities for single-ply and BUR membranes, asphalt strip shingles, and metal roofing were assumed to be 16 years, 15 years, and 25 years, respectively (Cash, 1997). Using unit costs from Means and other sources for tear-off, disposal, and replacement of the different roofing systems, the yearly roof replacement cost for all CONUS installations is estimated to be $99 million. This extrapolates to $133 million for the entire Army. Adding in costs for routine inspection and maintenance results in a total annual M&R requirement of approximately $200 million for roofing, which is equivalent to 13% of the total M&R dollars spent on Army buildings in FY96.

The "early" replacement of roofs has a significant impact on the Army's M&R requirements. Using 16 years as the acceptable service life, the 12-year service life of half of the membrane roofing systems on CONUS installations results in an estimated additional annual cost of $9 million. This does not include costs due to damage to furnishings, lost occupancy, mission interruption, and increased energy usage.

3 Army roofing management and improvement opportunities

From an Army perspective, roof management encompasses all activities necessary to ensure the suitable performance of roofing assets through their entire life cycle, including design and specification, construction quality management, and M&R management. Corps of Engineers (COE) District offices are responsible for executing the design and specifications, procurement, and quality assurance phases for new construction projects funded under the Military Construction, Army (MCA) program. District offices also provide these services for other customers, such as the Air Force, Army and Air Force Reserves, and the Veterans Administration. The roof designs for replacement projects and smaller building construction projects at Army installations are usually accomplished with the installation Directorate of Public Works (DPW) in-house design staff and Architect/Engineer (A/E) firms under contract. DPWs also manage the M&R of roofs on these installations.

3.1 Design

3.1.1 Process

Roof design has been recognized as one of several factors involved in ensuring satisfactory roof performance. Flaws and inconsistencies in the roofing system design and detail drawings can directly result in poor performance of a well-installed roof, and these problems can even impede proper installation by the contractor.

To accomplish building design, a typical COE District has an Engineering Division with an in-house design staff. This staff has several architects to handle the duties of developing roof designs, detail drawings, and technical specifications. Districts also maintain A/E contracts with design capabilities. An in-house designer or an A-E firm may accomplish the roof design tasks for a given project, depending on the District's current workload, expertise, and customer request.

As compared to COE Districts, the roof design process tends to vary more between installation DPWs. At some installations, roof replacement projects are accomplished under a roof requirements contract, which is awarded to a roofing contractor on an annual or multi-year basis for the purpose of expediting the process of accomplishing repairs and replacements. Typically, formal design and project specifications are not developed for these types of projects. Aside from being required to install a certain generic type of membrane and thickness of insulation, the contractor is allowed significant latitude in what he provides. The Army has limited assurance that it is receiving an adequately designed roofing system for the specific building.

Some Army roofing design inadequacies occurring on building construction projects can be attributed to the need to meet project cost and time constraints. For roofing as well as other types of construction, there is great pressure to keep costs down and to complete the contract documents in time to meet the project schedule. As a result, the design drawings may lack sufficient details to communicate the design properly, requiring the contractor to work out the deficiencies in the field. For many projects, the particular roofing system submitted by a contractor utilizes the membrane manufacturer's details, which may be very different from the project design details. Proper and sufficient review of these submitted changes and other similar activities may be difficult to accomplish without causing construction delays.

3.1.2 Improvement opportunities

Formalization of the engineering process through establishment of an internal roofing committee within each COE District can provide several functions to help improve the quality of roof designs. Committee responsibilities could include sharing expertise, performing final design review, and providing assistance to other parts of the organization on roofing matters. With representation from the design staff, construction quality assurance personnel, and users, the committee can serve as a focal point and conduit for ensuring that problems and issues are communicated back to the design staff.

To assure quality roof design from third-party contractors, A/E designers need to be pre-qualified. Aside from being a registered architect or professional engineer, a requirement for membership in IRWC (Institute of Roofing and Waterproofing Consultants International) would serve as evidence of both a professional license and roofing design experience. Membership with RCI (Roof Consultants Institute) could also be required. In addition to holding certain registration and membership, the A/E designer should be required to have experience in the design of roofing in the local area and climate of the project.

The Army should also consider the requirement of having all roof designs and specifications reviewed by an independent designer. This peer review of the designer of record's work would apply to both in-house and A/E designs. The small addition to the project cost would provide great benefit by avoiding serious mistakes that may be uncovered by the independent "second set of eyes."

The development of an Internet roofing site for the COE design and construction personnel was recommended during a 1997 Army roofing workshop. The major functions for the proposed site were to provide up-to-date information on Army and non-Army references, training seminars, and other roofing-related events, and to provide a means of communication for sharing lessons learned and obtaining expert assistance. Since that time, an existing Army Internet site for roofing maintenance (Roofing Information Support System) has undergone substantial improvements and enhancements. The site is not yet fully functional and requires some additional development work. When completed, it should provide an invaluable capability to the entire Army roofing community.

With replacements accounting for the large majority of the Army's current roofing projects, those procured through roof requirements contract methods may have a substantial impact on the Army's overall roof performance. Investigations are needed to determine the level and quality of designs used in such roofing projects.

3.2 Specifications

3.2.1 Process

Project specifications for military construction provide for the technical aspects of the work. As per Army regulations, Corps of Engineers Guide Specifications (CEGS) are used for the technical provisions of the project. These specifications are edited or "customized" by the designer to suit a project's particular requirements. They are coordinated with the construction industry and revised periodically to maintain state-of-of-the-art technology. Field users and others provide input for revisions through a standardized process.

These CEGSs, which exist for each of the major roofing systems, include three separate parts: General, Products, and Execution. The General section contains a listing of references and subsections with requirements for submittals, storage and handling of materials, fire and wind uplift resistance, and warranty. The Product section contains specific requirements for the materials and components used in the roofing system. The material and system installation procedures for the contractor are stipulated in the Execution section.

Within the CEGS, ASTM standards are referenced for the purpose of providing initial material property requirements. These standards are often prescriptive in nature and do not necessarily ensure their long-term durability. Although roofing materials and systems meeting the ASTM requirements have generally performed satisfactorily, their use has not been trouble-free. Examples of such problems include excessive plasticizer loss in the earlier poly(vinyl/chloride) (PVC) membranes and, more recently, shrinkage of ethylene-propylene-diene terpolymer (EPDM) membranes.

Because regulations and policy restrict the specification of brand names, the Army is required to specify roofing by material or system type. Therefore to ensure that satisfactory materials are specified and procured, standard test methods and performance criteria are needed which describe and measure how roofing materials will perform in service (Cash, 1993).

Even if good roofing materials are obtained for a project, the likelihood of getting an adequately performing roof is greatly increased if a reputable and capable contractor is awarded the contract. Typically, the Army's roofing specifications require that the contractor be a certified installer by the system manufacturer. This may ensure that the contractor has had at least some training on how to install the product. However, it must be realized that roofing manufacturers make greater profit by having more contractors use their product. Therefore, their certification process may be considerably less than rigorous. This problem is exacerbated by the fact that disreputable roofing contractors exploit the current government procurement system, which requires "free and open competition" and award to the lowest bidder.

3.2.2 Improvement Opportunities

As part of its Roofing Service Life Extension research program, USACERL is conducting studies to develop methods to predict the long-term performance and expected service life of different roofing membranes. The multi-year effort includes the following phases:

- Determining degradation processes and identifying the parameters of change.
- Developing performance models for the major generic roofing material types.
- Developing performance prediction models through the correlation with accelerated weathering.
- Establishing standard tests and service life criteria for roofing materials.

By having performance tests and service life criteria incorporated into ASTM standards and roofing systems selection and design guidance, the Army will be better able to ensure that appropriate roofing products are contracted for.

Aside from ensuring that quality materials are procured, a better means for qualifying roof installers needs to be pursued. For its own purposes, Factory Mutual (FM) has recently undertaken the development of a roofing contractor certification program. The draft standard is currently being reviewed by the roofing industry. It is intended to serve as approval criteria for certification of roofing contractors involved in the construction of roof assemblies approved by Factory Mutual Research Corporation (FMRC). It examines the training, education, and abilities of the contractor for the purpose of verifying that the installation of a roofing system will meet acceptable levels of performance and quality. A specification requirement for contractors to have such an approval can be instrumental in ensuring that good contractors are retained for Army roofing projects.

3.3 Construction quality management

3.3.1 Process

The Construction Divisions within COE Districts are responsible for construction management of COE-administered contracts and some DPW contracts. Per Army regulations, quality control (QC) is defined as the contractor's management, control, and documentation of its own activities to comply with the contract requirements. Quality assurance (QA) is the responsibility of the government, and is the system that ensures that the contractor's QC program is functioning properly and that the specified end product is realized.

For COE-managed projects, the field engineer's QA program includes a pre-construction meeting to discuss contract requirements, critical activities, design, and submittals required for the job. A "mutual understanding meeting" between the field engineer and the contractor is later held to discuss the submittal process. Prior to the start of roofing construction activities, the contractor must ensure that submittals as required by the technical specifications have been provided to the government and approved. The COE field office will receive the submittals, which for roofing may include FM and Underwriters Laboratories (UL) approvals, catalog cuts for materials, material specimens, and shop drawings of flashing and other details.

During actual construction, a three-phase control process (preparatory, initial, and follow-up) is utilized by the Corps Quality Assurance Representative (QAR) to ensure that the contractor is adequately conducting the required control processes. A single QAR is typically responsible for all aspects of a building construction project, ranging from foundation to roofing. This person may spend very little time on the roof or may be present throughout the workday. This will depend on the criticality of the ongoing work, other daily activities on the project, and available manpower.

For those roof construction projects managed by the installation DPW staff, the construction quality management process differs from the COE's and varies between installations. A Construction Inspection Branch within the DPW typically performs the quality assurance functions. The branch may have up to a dozen inspectors who are responsible for all types of construction and maintenance projects.

For both COE and DPW managed projects, the government's oversight function is lacking sufficient embeddment into the roofing QC process. The QC representative, an employee of the contractor, has the duty to ensure that contract requirements are met, and that workmanship is satisfactory. Giving the contractor full QC responsibility can best be described by the old adage "having the fox guard the hen house." The government is heavily relying on the specification and procurement processes to ensure that honest and reputable contractors are awarded projects.

An under-resourced quality assurance program only magnifies the shortcomings of the quality control process. The size of the existing staff compared with the size and complexity of their project load significantly limits QAR time spent on a construction activity.

3.3.2 Improvement opportunities

While having the contractor hire its own QC person is not the solution, having the manufacturer's representative perform these functions also presents problems. The manufacturer is in business with the contractors and does not want to alienate them. Having the Army hire an independent third-party roof monitor would provide the benefit of dissolving the relationship between the contractor and inspector, thereby strengthening the government's control of the process.

The contractor's best work quality can be expected when a roof monitor is present on the roof. An independent full-time QC inspector could also provide continual communication between the owner, the building occupants, and the contractor. A certification program, such as the Registered Roof Observer (RRO) operated by RCI, can serve to ensure that a minimum set of qualifications is met. With full-time third party QC expected to add only 2 to 4% to the roofing costs, the customer could easily be convinced of its benefits.

3.4 Maintenance and repair management

3.4.1 Process

Each installation DPW organizational structure has evolved over many years. As a result, the business processes used to achieve the maintenance of facilities differ widely among installations. Regardless of its structure, the DPW is required by regulation to perform annual inspections of its building roofs. In practice, some

DPWs perform inspections regularly, but this is the exception and not the rule. More often, crisis management, or in the case of roofing, leak management is practiced.

Installations that have routine inspection programs utilize teams of facility component inspectors to perform visual surveys of each membrane roof every 2 to 4 years. Building roofs that have significant problems are brought to the attention of management by submitting a work request. The major responsibilities for conducting detailed follow-up inspections and developing roofing repair and replacement requirements often reside with a DPW roofing manager or subordinate. This person may be a branch chief or senior engineer.

Installations that do not have formal inspection programs rely on their leak-management process to identify roofs having problems that require further investigation. Those roofs may be identified through work requests generated by the building occupants or submitted by maintenance shop workers after responding to a roof leak report. Neglecting roofs until leaks occur and require immediate attention is both disruptive and inefficient. Roof anomalies that could be resolved with minimal effort and cost go undetected until they become major maintenance and repair requirements.

A major reason why crisis management is perpetuated in a large organization such as the installation DPW is that its often easier to obtain funds from higher authority for emergency replacement than for routine inspection and maintenance programs. Just as "the squeaky wheel gets the grease," the leaky roof gets replaced because it has such a negative impact on the functioning of the building.

3.4.2 Improvement opportunities

Approximately 30 Army installations have implemented the ROOFER Engineered Management System (EMS) to assist them in managing the repair and replacement of their BUR and single-ply membrane roofing assets. The system provides a decision-support software application and standardized procedures for collecting inventory and inspection information, and evaluating roof condition. The DPW roofing managers use the program for data storage and analysis, and to generate management reports. This enables them to rate their present roof conditions, prioritize projects, and optimally allocate their roofing budget.

Installations, which have implemented ROOFER and actively use it, have been able to increase the benefit realized from their roofing M&R dollars. By having their roofing requirements quantified and documented through the use of standardized engineering procedures, Army roof managers have been much more successful in competing against other facility operation and maintenance (O&M) activities for limited repair funds.

The DPW roof managers that which do not actively use ROOFER need to be shown how investing in proactive roof management will enable them to stretch their roofing dollars further and improve the condition of their roofing assets. ROOFER developments need to continue as well. Current Army users have identified a requirement to extend the system from BUR and single-ply roofing to other types of roofing. These types include asphalt shingle, modified bitumen, and metal. Since the last revision of ROOFER in 1995, several new tools (e.g., a performance prediction model) and a stand-alone ROOFER geographical information system (GIS) application have been developed. These and other enhancements should be integrated into ROOFER to enhance its utility.

4 Summary

Roofing repair and replacement comprises a major portion of the Army's O&M budget. It is estimated that the Army incurs an added annual cost of $9 million due to "early" replacement of just their membrane roofing. Leaking roofs can have serious negative impact on a building's mission function and occupancy comfort and can result in additional costs from secondary damage to other building systems and contents.

The following list summarizes the recommendations for making improvements to the Army roofing assets management:

- formalized internal roofing committee
- enhanced pre-qualification requirements for A/E designers
- third party independent design reviews
- sustainment and enhancement of Army roofing internet site
- review of DPW roof requirement contract procedures and impact
- research studies to develop standard tests and performance criteria for roofing
- improved contractor certification requirements (proposed FMRC program)
- investigation of use of third party full-time roof monitors during construction
- continued development of ROOFER EMS and DPW training

Employing any of the recommendations by itself would not likely have a major impact on Army roofing performance. It must be remembered that good roofing requires proper design, materials, workmanship, and maintenance. Similarly, trying to solve the problem with just changes to business processes, application of existing technology solutions, or research and development is not the answer. By utilizing all of these and continually searching for new ones, the performance of Army roofing can be improved significantly over time and, in the end, can save millions of dollars.

5 References

Bailey, D., D. Brotherson, W. Tobiasson, and A. Knehans (1989), *ROOFER: An Engineered Management System (EMS) For Bituminous Built-Up Roofs*, USACERL Technical Report M-90/04.

Bailey, D., D. Simpson et al. (1997), *Statistical Analysis of ROOFER Database from 21 Army Installations*, USACERL Technical Report 97/83.

Cash, C., (1997), "The Relative Durability of Low-Slope Roofing", *Proceedings of the Fourth International Symposium on Roofing Technology*, Gaithersburg, MD, pp 119-124.

Cash C., and D. Bailey (1993), *Predictive Service Life Tests for Roofing Membranes: Phase 1*, USACERL Interim Report FM-94/03.

Headquarters, Department of the Army (HQDA) (1996), *Directorates of Public Works Annual Summary of Operations*, vol. I – Executive Summary, Fiscal Year 1996 (HQDA, Washington, DC).

R.S. Means, Inc. (1996), *Means Repair and Remodeling Cost Data – 18th Annual Edition Commercial/Residential*.

112 PREDICTIVE MAINTENANCE - WHAT SHOULD BE IN A CONDITION DATABASE

B.H. Hertlein

STS Consultants Ltd, Vernon Hills, IL, USA

Abstract

In recent years building owners and asset managers have begun to embrace the idea of Predictive Maintenance (PdM), but there is such a wide variety of tools being touted as "condition monitoring systems" that it is difficult for a non-PdM expert to make a wise choice. The owners and managers that have used PdM have experienced various degrees of success or failure, depending on how well the chosen methods met the purpose of the PdM program, and suited the particular structure on which they are being applied. This paper examines some of the requirements that monitoring and testing programs must meet if they are to provide reliable data that will be truly useful for PdM of concrete or masonry structures, and reviews some of the test methods currently available that meet those requirements.

Keywords: concrete, condition monitoring, corrosion, deterioration, nondestructive testing, predictive maintenance, PdM, repeatability,

1 Introduction

Predictive Maintenance (PdM) has for some time been a useful tool for production facility and utility supervisors striving to reduce life-cycle costs and find more efficient ways of using maintenance budgets. More recently building owners and asset managers responsible for concrete and masonry structures have also begun to understand how PdM can help them maintain the value or extend the service life of their structures. From bridge engineers to high-rise landlords to

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harbor masters there is now an appreciation of the potential benefits of condition monitoring and maintenance scheduling.

However, so many different test methods and survey approaches are being touted as "condition monitoring systems" by consultants and practitioners that it is difficult for a non-PdM expert to make a wise choice. The owners and managers that have used them have experienced various degrees of success or failure, depending on how well the methods met the purpose of their particular PdM program, and how appropriate they were to the specific structure on which they are being applied.

When selecting or planning a condition monitoring system, the first critical step for the owner or manager is to determine what the present condition and exposure of the structure is - the "baseline" data. That assessment will require an inspection and testing program that, in turn, will help identify which inspection or test methods provide useful data. If a test selected for the condition assessment proves to be of little value for that particular structure, it is better to find out at the beginning, rather than spend time and money on monitoring over several years, only to find that the data collected is useless.

The second critical step is to analyze the risks to the structure, and determine what information is actually required to monitor its condition. For this the nature and purpose of the structure must be considered, along with its present condition and environment.

The third step is to figure out the best way of getting the required information, and the form in which it should be collated and stored for ease of retrieval and analysis. In some cases none of these steps is easy.

The following text is not intended to be a definitive manual on the subject of condition monitoring, but rather to provide owners and managers with a better idea of some of the test and monitoring techniques currently available, and some key questions to ask the consultant who is proposing the monitoring program.

1.1 Baseline data - is it accurate and complete?

No amount of monitoring information will help you predict residual life and maintenance needs if you do not have a starting point, or baseline, to compare with. Establishing the baseline can be very difficult on an older structure, and is often not easy on a new one!

When establishing a baseline database, do not assume that everything is perfect in a new structure. Apparently minor deficiencies in the construction or finishing processes can create 'weak' spots - like Achilles heel, that leave your new structure vulnerable to the early onset of damage caused by deterioration or corrosion. For example:

- Misplaced reinforcing steel can lead to inadequate thickness of cover concrete
- Congested reinforcing steel can lead to poor consolidation of concrete
- Complex shapes in formwork can lead to poor consolidation of concrete
- Untrimmed tie wire and wire reinforcing chairs may be exposed at the concrete surface
- Wall ties may be missing in multi-withe masonry walls and facades
- Mortar debris caught on wall ties may span wall voids, providing a "bridge" for moisture
- Settlement and shrinkage can cause unexpected cracks that may not be readily visible on large or complex structures

The program of inspection and testing performed to gather the baseline data should be designed to thoroughly evaluate the condition of the structure, with no assumptions made about 'as-built' conditions. All data used in the condition database should be verified by direct observation or physical testing.

1.2 Is it repeatable?

For the condition database to be useful in predicting maintenance needs and/or residual life, the data gathered must be highly repeatable, otherwise comparison of 'then and now' conditions will be meaningless. The tests used to gather data for the baseline should be selected with repeatability in mind. Key considerations are :

- Does the test provide repeatable data, no matter who uses it?
- Does the test provide repeatable data no matter what the weather and ambient conditions?
- Will the test be able to provide valid data as the structure or surface starts to deteriorate?

This emphasis on repeatability includes visual observations made by the inspecting personnel. Visual assessment is inherently subjective. Any data gathered by observation should be supported by some form of quantification, or by photographic or videotape record. For example, an assessment of the amount of cracking in an asphalt pavement, or efflorescence on a basement wall should be accompanied by a photograph or accurate sketch that includes a readily identified scale to give a sense of proportion. Such scale might be provided by the number of bricks in a wall, an adjacent door or steps, or a tape measure or notebook held adjacent to the feature being recorded.

Similarly, the conditions under which photographs are taken should be recorded, particularly where color is an important indicator of a condition. The time of day, use of flash, or use of incandescent lighting, will all affect the apparent color tones in a photograph or video.

1.3 Is it relevant?

Recent advances in computers and data acquisition technology have made it so easy to collect and process large amounts of data that sometimes quantity is substituted for quality. The data collected must be relevant to the purpose of the monitoring program, and to the specific structure that is being monitored. There is no "one-size-fits-all" monitoring program.

Consider the example of a storage silo that is several hundred miles from the ocean and is not exposed to deicing salt spray from highways. If the initial condition assessment of shows low chloride content in the concrete there is little point in monitoring chloride penetration. However, if the same silo is adjacent to a fossil-fueled boiler plant in a factory or electricity generating station, condensation of flue gases could introduce chlorides, and the associated likelihood of accelerated reinforcing steel corrosion.

Monitoring of vibrations is often recommended, based on the theory that the natural, or resonant frequency of the structure will change as deterioration occurs. However, experience has shown that on certain structures the damage must be significant before any measurable change in natural frequency occurs. Such information is therefore not much use in a program that is intended to predict maintenance needs and maximize the structure's life.

2 Physical tests

There are several nondestructive or semi-destructive tests that can provide reliable baseline data, and are repeatable enough to be useful in monitoring or tracking symptoms of deterioration in concrete and masonry structures. Nondestructive testing in the context of this paper means a test that does not mar the surface of the test area, or require removal of a sample of material.

Semi-destructive means that some surface marking may result from the application of the test, or the removal of a small sample of material, but the integrity and durability of the structure will not be affected by the test.

Space limitations preclude lengthy descriptions of the test methods here, but for those seeking more information, a very useful reference that gives more detail on all of the test methods mentioned in this paper is Report No. ACI-228.2R, published by the American Concrete Institute (ACI, 1998) In addition, all of the methods shown with an ASTM designation are described in the ASTM Annual book of standards, Volume 4.02 - Concrete and Aggregates. (ASTM, Annual)

2.1 Carbonation depth

Fresh concrete has a pH of about 13.5. It is this alkalinity that inhibits corrosion of the embedded reinforcing steel by forming a passive, corrosion resistant coating. Over time carbon dioxide in the air combines with moisture to form dilute carbonic acid, which reacts with the alkaline constituents of the cement paste, gradually reducing the pH of the material. Eventually the pH of the concrete is reduced to such a level (less than about pH 11.5) that it can no longer maintain the passive coating on the reinforcing steel, and corrosion can ensue.

The carbonation process originates at the surface of the concrete. By measuring the depth of cover concrete over the reinforcing steel, and monitoring the rate of penetration of carbonation, the time it will take for the carbonation front to reach the reinforcing steel, and increase the risk of corrosion, can be estimated.

Accurate measurement of carbonation depth can only be made from laboratory examination of a sample of the concrete, but there is a simple field test that can give a very useful approximation of carbonation depth. A small diameter hole is drilled into the concrete, and the edge of the hole is flaked off with a small chisel to create a freshly broken surface. A chemical indicator is sprayed on the freshly broken surface. The simplest indicator is Phenol Phthalein, which is clear when applied, but turns purple when in contact with uncarbonated concrete. A more sophisticated "rainbow" indicator changes color according to the alkalinity of the concrete, allowing an estimate of approximate pH value to be made.

It should, however, be borne in mind that this field test only indicates the depth of the fully carbonated concrete. Partial carbonation may have advanced further than the depth indicated by the field test.

2.2 Concrete cover to reinforcing steel

If the probable onset of corrosion is to be predicted from carbonation depth measurements or penetration of externally-originating contaminants such as chlorides, then the depth of concrete covering the reinforcing steel must be known. Several proprietary instruments, or "cover meters", are available for estimating the depth of cover concrete nondestructively.

The term "estimating" is used because the accuracy of the depth measurements made by these instruments is affected by the type of reinforcing steel, the presence of magnetic or metallic aggregates, and the orientation and proximity of the steel bars and other embedded metal items such as conduits or pipes.

The accuracy of depth estimates for a specific structure can be improved by calibrating against direct physical measurement of reinforcing steel depth and size at a location where the steel can be exposed drilling or cutting. Complications will arise where bars are closely spaced, and the meter may not be able to discriminate a single bar for reliable measurement of depth.

Strong electrical or magnetic fields will also affect the accuracy of the measurements, and in extreme cases may make the instruments so unstable that no useful information can be obtained.

Because of these physical influences, the accuracy of the cover meter is partly dependent on the operating principal of the particular meter used. A report published in 1992 by the United States National Institute of Standards and Technology (Carino, 1992) examined the various types of cover meter then available, and documented the relative accuracy, discrimination and sensitivity of each instrument.

2.3 Ultrasonic pulse velocity (ASTM C-597)

The velocity of an ultrasonic pulse through concrete is a function of the modulus and density of the concrete. Ultrasonic Pulse Velocity (UPV) measurement is therefore useful in assessing concrete quality and uniformity. The method can locate areas of lower density material resulting from construction defects such as poor consolidation (honeycombing) and microcracking associated with freeze/thaw or fire damage, or alkali aggregate reaction. UPV can provide useful data for monitoring deterioration over time.

However, concrete has an inherent degree of variability that must be considered. The UPV calculated for a given test is the average velocity for each particle along the transmission path. If the UPV transmitter or receiver are moved a few millimetres across the surface, the pulse will probably pass through a different set of particles. The average velocity will vary depending in aggregate size, orientation, and distribution. Where it is required to monitor deterioration by repeating UPV tests over an extended period of time, it is essential that the precise locations of the transducers be marked or documented, and accurately repeated. Experience has proven that variations in transducer locations can cause variations in calculated velocity that render the data useless for trend tracking. (Hertlein, 1992)

2.4 Impact/Echo test (ASTM C-1383)

The impact echo test uses a BB sized impactor to strike the material surface, generating an elastic wave that radiates from the point of impact. The wave is reflected back towards the surface by discontinuities in the material. A transducer mounted on the surface close to the impact point detects the reflected waves. A data acquisition system records the transducer data, from which the depth to the reflecting surface or discontinuity can be calculated.

The method can measure plate thickness from a few millimetres to about a metre or so with a high degree of accuracy. The Impact/echo test can be useful in checking the uniformity of grout in post-tensioned tendon ducts. The method has also proven useful in identifying debonding or delamination of concrete repair materials. (Henriksen, 1995)

The small size of the impactor, and the limited range of the displacement transducer, make the Impact/echo method unsuitable for use on roughened or spalled surfaces. (Davis et al, 1998) Where surface roughness is excessive, grinding of the concrete is required to provide a suitable test surface. The method is also hampered by thick elastomeric coatings.

2.5 Impulse response test

The Impulse Response Spectrum test is also sometimes known as the Sonic Mobility test. The structure is instrumented with a velocity transducer and excited by an impact from a small sledge hammer which contains a load cell to measure force input. The data from both instruments is recorded and both signals are converted into frequency-based data, where velocity is divided by force to provide a normalized response of mobility against frequency, or the Impulse Response Spectrum (IRS).

The IRS contains information on the dynamic stiffness of the test point and the quality of the concrete. The integrity of the structure can be assessed from IRS data, and the test can identify conditions that are not visible to the eye, such as sub-surface honeycomb, delamination, micro-cracking, and lower modulus concrete. (Davis and Kennedy, 1997)

The IRS test is unaffected by surface moisture or paint, but thicker elastomeric coatings can deform at the impact point, leading to variability of stiffness measurements. (Hertlein et al, 1995)

2.6 Chloride content analysis (AASHTO T260, 1995)

Free chloride ions from deicing salts or seawater penetrate concrete and masonry rapidly, where they become a catalyst in the corrosion of embedded steel such as reinforcing or conduit. Concrete that is heavily contaminated with chlorides cannot be effectively repaired without first removing all the contaminated material. The effects of highway de-icing salts on bridges and parking decks is common knowledge. What is less well-known is that flue gases from fossil fueled boiler plants can contain a high proportion of Hydrochloric Acid (HCl) vapor, which, at normal operating temperature, is discharged with relatively little harm to the flue or chimney system. However, if the temperature of exposed steel or concrete surfaces is below the dew-point of HCl, the acid condenses and directly attacks the material.

Chloride content can be determined in a laboratory from a drilled powder sample of concrete or masonry. The rate of chloride penetration and the direction from which contamination is occurring can be determined from samples taken from different depths in the same drill hole.

2.7 Half-cell potential/resistivity (ASTM C876) and rate of corrosion

Corrosion of reinforcing steel in concrete is an electrochemical process that requires oxygen, water, and a conductive medium through which electrical current can flow. Dry concrete has a high resistivity and impedes corrosion currents. Moisture or contaminants in the concrete can lower its resistivity until it becomes conductive enough for corrosion currents to flow through it.

The differences in electrical potential that are needed to drive the corrosion currents can result from forming and working stresses in the steel, and surface contaminants. The potential differences and the resistivity of the concrete can be measured to provide an indication of where corrosion activity is likely, or is occurring. Higher potential differences, and lower concrete resistivity increase the likelihood of corrosion activity.

The Half-cell Potential method is a means of measuring the potential difference between various points on the reinforcing steel. An electrical half-cell, made of silver in silver chloride (Ag/AgCl), is connected in series with a sensitive voltmeter, and a reference location on the reinforcing steel. A measurement circuit is completed by pressing the tip of the half-cell against the concrete surface over another part of the reinforcing steel. The potential difference (PD) between the test point and the reference point is then measured and recorded.

A fundamental weakness of the method is that the conductivity of the concrete will vary as moisture content changes, so changing the apparent PD value measured. Similarly, electrical discontinuities in the reinforcing armature, or poor contact between the half-cell and the concrete surface, will affect the test results. Interpretation of test results can be made more reliable if PD data is correlated with measurements of the concrete resistivity taken at the same time. Poor correlation between PD and resistivity values indicate invalid test results due to one or all of the factors given above.

Since ambient conditions such as humidity and temperature will affect the measured values, PD/resistivity data should not be interpreted in terms of absolute values. More reliable estimates of the probability of corrosion activity are made from assessing the difference, or gradient, between adjacent parts of the structure. (Clemena et al, 1992)

The Rate of Corrosion test is based on the same principle as the Half-cell Potential method, but measures the current that must be impressed on the concrete/reinforcing steel circuit to maintain a certain level in potential difference, or polarization. It is therefore affected by the same variations in moisture content and contaminants in the concrete. These tests can only provide repeatable data if ambient conditions are identical for each set of measurements. (Clemena et al, 1992)

A common alternative half-cell construction is the copper/copper sulfate ($Cu/CuSO_4$) half-cell, but this type of cell is adversely affected by any chlorides that may be present on the surface of the concrete. The Cu/CuS_{O4} cell is therefore not suitable for parking decks or bridges that may be contaminated by deicing salts. Chlorides are also present in abundance in the flue gas from most coal burning plants. Investigation of concrete chloride content should therefore be performed prior to using $Cu/CuSO_4$ Half-cells on chimneys and nearby structures.

If reinforcing steel has not already been exposed by spalling of the concrete, access for the reference connection to the reinforcing steel must be provided by drilling or chiseling a small hole in the concrete. The half-cell potential test cannot be used on reinforcing steel coated with epoxy or any other non-conductive material.

3 Instrumentation and embedded monitors

3.1 Crack monitoring

Monitoring of cracks to measure growth and/or movement can provide valuable information for condition assessment and durability prediction, and in some cases can help determine the cause of cracking. The crack can be monitored periodically or continuously. Periodic monitoring can be done very simply by installing a mechanical or optical measurement system, in which reference points or grids are fixed to the structure either side of the crack.

If continuous monitoring is required, or the crack is difficult to access on a regular basis, monitoring can be automated by installing strain gages and a suitable recording system. However, strain gages can accommodate only a limited range of movement, typically only a few millimetres at most, and should be selected with care if large crack movements are anticipated.

If movements are anticipated to be too large for continuous monitoring by strain gages, then displacement transducers could be considered. They generally will not have the fine resolution of strain gages, but can be made to accommodate movements of 100 mm or more.

Since structural dimensions will vary as temperature changes cause thermal expansion or contraction, the temperature should be measured when the strain or crack width is measured. Correlation of temperature variations with strain or crack width measurements can provide clues as to the cause of the cracking, and the behavior of the crack. Similarly, on tall or exposed structures, measurement of wind speed and direction can be useful.

3.2 Embedded instrumentation

There are a number of transducers available today for embedment in the concrete or masonry of the structure to monitor various parameters. Measurements of acoustic emission, chloride contamination, displacement, humidity, temperature, strain, and vibration can all add valuable data to a PdM database.

A Canadian firm has developed an instrumentation grid for post-tensioned structures that "listens" for the characteristic sound produced by a breaking tendon. Using a computerized triangulation process that analyzes the strength of the vibration impulse generated by the tendon break, the system can identify where in the structure the break occurred. (Elliot, 1996)

Key elements of certain structures, such as bridge cables, can be instrumented to monitor the natural frequency of vibration. Significant changes in the integrity of the element, or the load distribution in the structure, can often be detected by a shift in the natural frequency of the element. Care must be exercised in this application however, because, as mentioned earlier, some structural shapes and materials have to be severely damaged before any significant change in natural frequency occurs.

Structural protection systems can be monitored to provide warning of failure. Monitoring the supply current in impressed-current cathodic protection systems is an obvious choice. Other protection systems are often overlooked. For example, a sudden decrease in resistivity, or an increase in chloride content can indicate the failure of a parking deck or bridge deck membrane. University researchers have recently reported the development of a fiber-optic sensor for chlorides that can be permanently embedded within the concrete deck of a bridge or parking structure. (Fuhr et al, 1998)

An increase in relative humidity can indicate an increase in moisture content associated with the failure of a wall coating or water-proofing membrane. Again, the sensors can be permanently embedded within the structure.

Mechanical services can be monitored too. An increase in vibration in a drawbridge or elevator motor room may be a symptom of structural problems, or more likely, mechanical problems that may eventually cause structural problems. Many large buildings and industrial plants already have trend-tracking vibration monitoring programs in place for the mechanical equipment. Some of these systems may be readily modified to include information from transducers located in or on the building structure.

4 Summary

The concept of "smart" structures has been bandied about for some time. Long term monitoring of almost any structural or behavioral parameter by a reliable, repeatable measurement will eventually show a regular pattern that may vary on a daily, monthly, or annual basis - but once the pattern has been established, deviations from it will be readily recognized. There are enough commercially available systems to make the idea of a smart structure an economical reality. Integrated monitoring systems can report continuously on key parameters that can be compiled into a database, and thus provide warning of changing conditions that may need immediate attention, or simply need to be accounted for in next year's budget.

For existing structures a full retrofit with permanent monitoring equipment may not be economically viable, but a carefully considered program of inspection and testing can gather vital information, provided that the methods chosen acquire repeatable data, and key environmental variables, such as temperature and humidity, are included in the measurement program.

A well-designed test and inspection program can provide reliable, repeatable information that will help the owner or manager plan maintenance needs proactively.

5 References

AASHTO (1995) *Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials - T-260* American Association of State Highway and Transportation Officials, Washington, DC

American Concrete Institute (1998) *Nondestructive Test Methods for Evaluation of Concrete in Structures ACI Report 228.2R,* American Concrete Institute, Farmington Hills, Michigan

ASTM - *Annual Book of ASTM Standards, Volume 4.02 - Concrete and Aggregates* American Society for Testing and Materials, West Conshohocken, Pennsylvania

Carino, N. J. (1992) *Performance of Electromagnetic Covermeters for Nondestructive Assessment of Steel Reinforcement- NISTIR 4988* National Institute of Standards and Technology, Gaithersburg, Maryland

Clemena, G. G., Jackson, D. R., and Crawford, G. C. (1992) *Benefits of using Half-Cell Potential Measurements in Condition Surveys of Concrete Bridge Decks* Paper No. 920248, TRB 71st Annual Meeting, January 1992. Transportation Research Board, Washington, DC

Clemena, G. G., Jackson, D. R., and Crawford, G. C. (1992) *Inclusion of Rebar Corrosion Rate Measurements in Condition Surveys of Concrete Bridge Decks* Paper No. 920344, TRB 71st Annual Meeting, January 1992. Transportation Research Board, Washington, DC

Davis, A. G., Hertlein, B. H., Lim, M. K., and Michols, K. A. (1996) Impact-echo and impulse response stress wave methods: Advantages and limitations for the evaluation of highway pavement concrete overlays. *Proceedings of Conference on Nondestructive Evaluation of Bridges and Highways*, Scottsdale Arizona, Dec. 1996., Society of Photo-optical Engineers (SPIE), Bellingham, Washington

Davis, A. G., and Kennedy, J. (1998) Impulse response testing to evaluate the degree of alkali-aggregate reaction in concrete drilled shaft foundations under electricity transmission towers. *Nondestructive Evaluation Techniques for Aging Infrastructure and Manufacturing*. San Antonio, Texas, April, 1997. Society of Photo-optical Engineers (SPIE), Bellingham, Washington

Elliott, J. F. (1996) Monitoring prestressed structures. *Civil Engineering July, 1996*. American Society of Civil Engineers, Reston, Virginia

Fuhr, P. L., Huston, D. R., and MacCraith, B. (1998) Embedded fiber optic sensors for bridge deck chloride penetration measurement *Optical Engineering, Vol. 37, No. 4, April 1998*. pp. 1221 - 1228. Society of Photo-optical Engineers (SPIE), Bellingham, Washington.

Henriksen, C. (1995) Impact-Echo testing *Concrete International May 1995*, pp. 55-58. American Concrete Institute, Farmington Hills, Michigan

Hertlein, B. H. (1992) Learning to love NDT. *Civil Engineering January 1992*, pp. 48-50. American Society of Civil Engineers, Reston, Virginia

Hertlein, B. H., Johnson, M., and Michols, K. (1995) *Condition Assessment of Chimneys and Other Structures using Non-Destructive Testing* American Power Conference, April 1995, Chicago, Illinois

113 EFFECTIVE ROOF MANAGEMENT — UNDERSTANDING THE LIFE CYCLE OF YOUR ROOF SYSTEMS

C.P. Hodges

Facility Engineering Associates, Fairfax, VA, USA

Abstract

Roof system management can be an effective tool for keeping track of selection, maintenance, and long term performance of roof systems. The National Roofing Contractors Association currently lists over 1,300 different roof membrane products and systems that are available to building Owners. On any large facility, it would ***not*** be unusual to find dozens of different types of roof systems in use. Each roof system has its own maintenance requirements and life cycle performance characteristics. This presentation is oriented toward reviewing available roof life cycle data and management systems, and suggesting some key elements that should be included in a roof management system. An effective roof management system can also utilize past performance, material properties, and life expectancy databases to sort through and select the optimal systems for the next re-roofing project. This presentation will review available life expectancy data for different types of roof systems, and summarize key factors that have a large influence on life expectancy. Much of the data presented is derived form actual surveys and life-cycle predictions from several commercially available roof management software packages, and published studies on expected service life for many different types of roof systems.

Keywords: roof system management, life cycle, maintenance, material properties, inspection systems, prioritization

1 The roofing industry

The National Roofing Contractors Association (NRCA) closely tracks trends in the roofing industry. NRCA's 1997-98 Market Survey [1] predicted a $21 billion market for roofing in the United States in 1998. About 75% of the $21 billion was

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projected to be in the replacement of existing roof systems. The NRCA also publishes the "Commercial Materials Low-Slope Guide" on an annual basis. The purpose of the guide is to maintain a list of currently available low slope roof systems. The guide lists roof membrane systems by type, insulation types by manufacturer, and available warranties. Roof membrane systems are listed under the following categories:

- ☐ Built-up roofing Membranes
- ☐ Modified Bitumens
- ☐ Single-Ply Roof Membranes
  - Vulcanized Elastomers
  - Nonvulcanized Elastomers
  - Thermoplastics
- ☐ Spray-applied Polyurethane Foam Roof Systems
- ☐ Metal Roof Panels

The 1998 edition of the low-slope guide lists over 1300 available roof systems for new and replacement roof systems. This represents a seven-fold increase over the number of membrane systems listed in the first edition of the guide in 1983. In the chart below, the first series represents the number of roof systems available (by type) in 1983. The second series represents the number of available systems (by type) as listed in the 1998 edition of the guide.

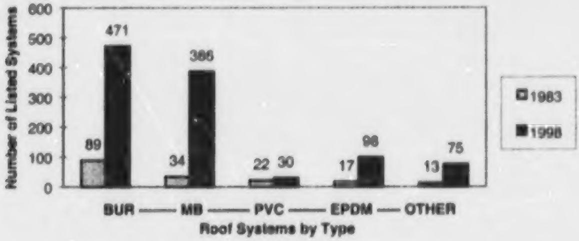


Fig. 1: Roof system availability – 1983 to 1998

Since the early 1990's the guide has been expanded to represent several new categories of roof systems that were not represented in 1983. Figure 1 does not include two of the guide's newest categories of roof system, spray applied polyurethane foam, and metal roof systems. The 1998 edition of the guide lists 131 spray applied polyurethane foam roof systems and 155 metal roof panel systems.

This tremendous rate of growth in the roofing industry has lead to the introduction of an unprecedented number of new products. The last 20 years in the roofing industry has been a time of new product development and implementation. Unfortunately, not all roof systems have passed the test of time. Along with significant growth, we have seen instances of product failure and inability of specific formulations of products to resist weathering. For example, some early formulations of unreinforced polyvinyl chloride (PVC) sheet material in loose-laid ballasted membranes were prone to shattering due to loss of plasticizer [2]. Degradation due to the exposure of chlorosulfonated polyethylene (CSPE) based single-ply roof membranes to ultraviolet (UV) has been documented [3]. Binder erosion in the CSPE membrane can lead to the development of cracking down to the level of the reinforcement, and the development of pinholes in the membrane.

Premature failure of roofing systems cannot always be attributed to product failure. C. W. Griffin, P.E., in the "Manual of Built-up Roof Systems" cites several causes of premature failure; the difficult environment in which a roof must perform, an abundance of new materials, complex roof system design, increasing roof dimensions, application issues, and trends toward more flexible buildings [4].

The issue of life expectancy of any given roof system is often debated and rarely agreed upon. We know that due to the forces of nature, a roof system will deteriorate. The stresses imposed on a roof system include thermal cycling, ultraviolet radiation, snow load, wind load, traffic, and a variety of other forces that vary by region. Each roof system will have a characteristic deterioration curve, such as the one shown in Figure 2.

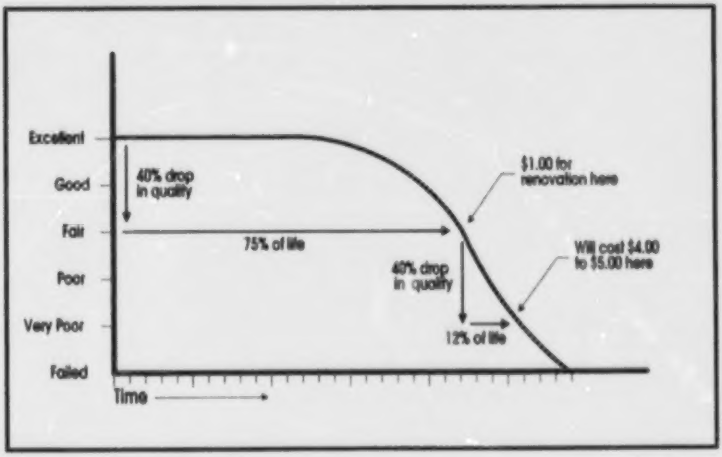


Fig. 2: Roof deterioration curve

Roof system failure is rarely a catastrophic event. The most common mode of failure is an increase in the occurrence of leaks. This is accompanied by an increase in the amount of annual maintenance dollars spent to maintain a roof in the last few years of its service life. In modeling the life cycle of a roof system, we generally assign a service life, then extend that service life by three to five years, assigning increasing annual maintenance costs during the extended period. This provides a more realistic economic forecast of true cost. The life cycle curve shown above is an illustration of the rapid decline in the condition of a roof system over the last few years of its life cycle.

2 Roof life cycle

With few exceptions, roofs fail over a period of time that usually lasts several years. As a roof nears the end of its life cycle, maintenance cost rises as owners respond to increasing instances of leaks. Failure of a roof system is usually defined as that time when replacement becomes necessary to protect the building. Until the past several years, there has been little published data regarding the life expectancy of a roof system. Factors such as the length of time required to conduct a study, and the dozens of variables that effect service life make life cycle prediction difficult.

In 1997, Cash [5] presented the results of a 1996 nationwide survey of durability, initial maintenance, and life cycle cost for frequently used low-slope roof systems. One of the results of the study is summarized in Table 1, Roof Life Cycle Costs. Cash notes in the study that durability is not represented by one value, rather a range of values. The mean durability of several roof systems is listed in the Table below.

Table. 1: Roof Life Cycle Costs for low-slope roof membrane (Cash (1997))

Membrane Type	Installed Cost $\$/ft^2$	Disposal Cost $\$/ft^2$	Mean Durability (Years)	Maintenance Cost $\$/(ft^2 \times yr.)$	Life Cycle Cost $\$/(ft^2 \times yr.)$
Coal tar-glass felt & pitch built-up	3.23	1.12	21.9	0.10	0.30
Asphalt-glass felt & asphalt built-up	2.28	0.81	16.7	0.12	0.31
Asphalt-glass felt & pitch built-up	2.87	1.07	17.7	0.09	0.31
Coal tar-organic felt & pitch built-up	2.97	1.10	23.0	0.14	0.32
EPDM (Rubber)	2.21	0.98	14.2	0.10	0.33
Asphalt-organic felt & asphalt built-up	2.27	0.86	14.7	0.12	0.33
SBS-Modified Bitumen	2.70	0.93	15.9	0.11	0.34
APP-Modified Bitumen	2.35	0.72	13.7	0.12	0.34
Polyvinyl Chloride (PVC)	2.54	0.84	13.8	0.11	0.36
Metal	4.94	1.27	25.0	0.11	0.36
Ethylene Propylene, Other	2.61	0.73	12.7	0.11	0.37
Reinforced Hypalon, CPE	2.69	0.75	12.8	0.11	0.38
Polyisobutylene (PIB)	2.76	0.76	10.6	0.09	0.42
Polyurethane Foam*	2.57	1.27	12.1	0.15	0.47

* Includes Insulation

Studies such as those by Cash have alluded to the fact that durability (life cycle) may be heavily dependent on heat aging (thermal load) of roof systems. Cash hypothesizes that roofs exposed to higher temperatures over their life cycle experience decreased durability over those that do not experience the same thermal loads. In other words, the mean time to failure decreases as the thermal load increases. Other factors which may have a significant effect on life cycle include slope, roof traffic, and maintenance.

Data presented by Kyle and Kalinger [6] at the Fourth International Symposium on Roofing Technology showed several examples of reductions in service life due to maintenance practices and roof traffic. In some cases, the reductions were over one-fourth of the service life between under-maintained roofs and highly maintained roofs. Reductions in service life of 18 to 20 percent were cited for heavily trafficked roofs versus those that sustained minimal roof traffic.

Most designers would recognize that roof slope plays a critical role in extending the service life of a roof system. However, there is very little published data available to compare roof system service life versus slope. Clearly, more study is needed to evaluate the effect of roof slope on service life.

Table 4 represents the results of a study of average roof life from a database of over 24,000 roof systems studied from 1982 to 1996. The study was conducted by Schneider and Keenan [7].

Table 2: Average life of roofing system by type (Schneider and Keenan (1997))

ROOF SYSTEM	1982 AVG LIFE	1994 AVG LIFE	1995 AVG LIFE	TO DATE AVG LIFE	TO DATE AVG LIFE
Asphalt BUR	13.3	13.6	13.6	13.6	13.64477
Coal Tar BUR	18.2	18.1	18.1	18.1	18.08732
Modified Bitumen	16.9	17.2	17.3	17.3	17.33041
Polyvinyl Chloride	9.1	9.4	9.4	9.5	9.48136
EPDM Adhered	17.7	17.9	17.9	17.9	17.94122
EPDM Ballasted	17.7	18.3	18.4	18.4	18.43481
EPDM Mech. Fast.	17.7	16.8	16.8	16.8	16.83660
Metal	17.9	22.4	22.4	22.4	22.44841
Shingles		12.2	12.2	12.2	12.23594

A comparison of the study by Cash to the study by Schneider & Keenan indicates that there is general agreement regarding the relative position of the service life of different types of roof systems in relation to each other. However, the service lives of several roof systems types is shorter in the Schneider & Keenan study, most notably in the case of built-up, PVC and metal roofing. The average service life of modified bitumen and EPDM roofs is longer in the Schneider & Keenan study.

The Schneider & Keenan study also appears to indicate that there has been no significant increase in service life for any of the roof systems in the study over the period of 1982 to 1995. This may be an indicator that although there may have been technological improvements in several of the newer types of roof systems since their first introduction, we have yet to see an increase in service life due to improved technology. Some data, such as that presented by Hoff [8] at the Fourth International Symposium on Roofing Technology, indicate that technological improvements in EPDM seaming, flashing, and application techniques have significantly reduced annual repair and maintenance expenditures on roofs constructed in the late 1980's as compared to those constructed in the early 1980's.

3 Managing roof systems

If a facility has more than four or five buildings, chances are there will be multiple types of roof systems present. In a campus setting of ten or more buildings, it would ***not*** be uncommon to have each major roof type represented in the inventory. Chances are also very good that at least one roof replacement is required in the next several years. The service life of a replacement roof system can vary from 10 to 25 years. At an average for removal and replacement of an existing roof system of $4.00 to $8.00 per square foot, the amount of money spent replacing roof systems can be significant. The expenditure of several hundred thousand dollars per year for roof replacement is not uncommon for large facilities.

Annual roof maintenance cost is also an important consideration. The level of roof maintenance has a significant impact on the annualized cost of installing and maintaining a roof system. The Cash data indicates that the average annual maintenance cost for the roofs reported in the study are typically one-third of the average annual life cycle cost. It has been the authors' observation that a lack of annual maintenance can lead to a shortening of expected service life. From a cost standpoint, the increase in annual life cycle cost for an under-maintained roof can far outweigh the cost savings from ***not*** performing the maintenance. The Kyle & Kalinger study presents data that supports the hypothesis that a well maintained roof may have a significantly increased service life over one that is poorly maintained.

With a significant amount of dollars at stake, it becomes cost effective to implement a roof management program. At the very least, roof system management can become an integral part of an overall facility management program.

4 Roof management systems

Many facilities are managed on a reactive basis where roof systems are concerned. When a roof leaks and becomes a constant drain on annual maintenance dollars, it is programmed for replacement. An effective roof management system can help quantify roofing assets, and program maintenance and replacement costs on an annual basis. A roof management program can help facility managers reduce annual maintenance cost. The savings is realized by programming the appropriate repairs at the right time in the life cycle, thus extending the life cycle curve shown in Figure 5.

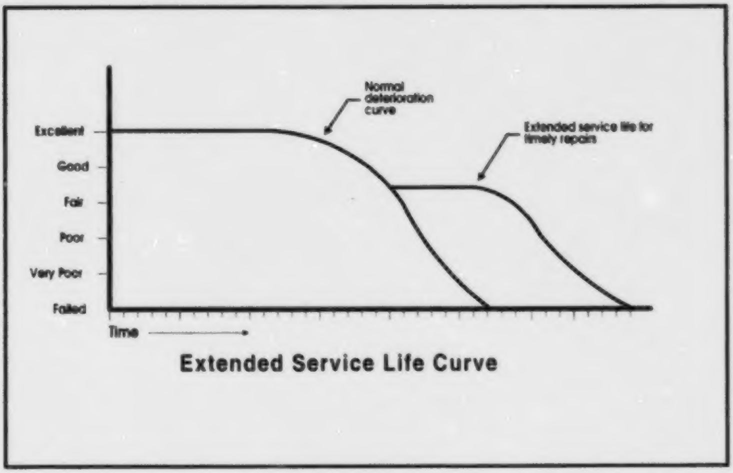


Fig. 5: Extension of life cycle

The first step in an effective roof management program is an accurate quantification of existing roof assets by periodic inspection. This includes gathering basic roof system information such as roof type, age, and maintenance history. This information can be entered into a simple database, or into a facility management program. More sophisticated roof management programs include quantification of roof defects and compilation of a roof rating. The rating can be used to compare to other roof systems in the database and help in the prioritization of repairs or replacement. The roof management system can also be linked to a construction repair and replacement cost database, to derive one-time or annualized repair costs. Several of the most detailed roof management programs include each of these elements, plus the ability to conduct life cycle cost analysis on various roof replacement options. A reasonable service life prediction can be made for each potential roof replacement type, and comparisons made to other types of roof systems. If the cost and potential service life is known, a reasonable life cycle cost analysis can be made. There are two critical elements in making this type of comparison worthwhile: (1) An accurate estimate of service life is made, and (2) Accurate replacement costs are used.

Unfortunately, there is very little empirical data regarding the expected service life of a given roof system. The data that is available in the industry is often proprietary to the interest of the manufacturer of the system, and varies widely within the industry.

The key elements of a roof management program include the following:

- ☐ An ***inspection system***
- ☐ A ***database*** to manage roof information and inspection data
- ☐ Repair and replacement ***cost data***
- ☐ Reasonable ***service life predictions*** for various roof systems
- ☐ ***Capability to prioritize*** repairs and replacements
- ☐ ***Reporting capabilities*** useful to the facility manager

Effective roof management can start with the implementation of a basic inspection system. This does not need to be a costly or time-consuming process. Collecting data on roof type, age, and general condition can be a useful first step.

The more sophisticated the management system, the more detailed the inspection information required. This information is input to a detailed database that makes retrieval and comparison to other roof systems of facilities easy. The most detailed management systems include cost data, service life prediction, prioritization, and reporting capabilities tailored to a Facility Manager's financial and capital needs programming requirements.

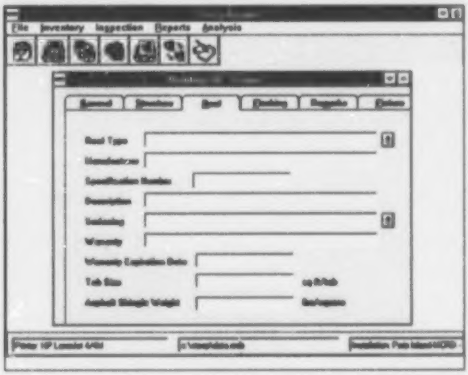
5 A roof management example

There are many commercially available roof management systems. Many are proprietary and can be costly to purchase or to implement. One of the most detailed and commonly used roof management tools is the U.S. Army Corps of Engineers ROOFER® Roof Management System (RMS). The basis of the ROOFER® RMS is the collection of detailed inspection information and development of a Roof Condition Index (RCI). The RCI is based on a sum of the quantity and density of defects found in a roof system. The RCI is derived as a composite of three separate indices developed for the roof membrane, flashing, and insulation.

The overall RCI is numerical value (from 0 to 100) which defines the roof condition. The numerical value can be used to compare to other roofs (to aid in prioritization), or can be used as a decision making tool as to when a roof requires replacement. The ROOFER® RMS has cost estimating capabilities for repairs or replacement. This allows for economic cost comparison of *repair vs. replacement* scenarios for any given roof, as well as comparison between roofs.

The ROOFER® RMS has been developed to include built-up, modified bitumen, and single-ply roof systems. One attractive feature of the ROOFER® RMS is that the cost of the system is extremely low since it was developed in the public domain. One drawback to the ROOFER® RMS is that since it utilizes detailed inspection information, the initial inspection cost may be significantly higher than other more basic systems.

Depending on facility type, it may also be useful to implement a roof management system on steep roofs. Steep roofs are generally those with slopes of at least two inches per foot, and include slate, tile, asphalt shingles, metal panels, and a variety of composite and man-made coverings. The following is an example of a data screen from a steep roof management program.



This program uses similar types of detailed roof inspection information as the ROOFER® RMS to develop a roof condition index. The implementation of these types of roof management programs can now be used for all of the major roof system types; built-up and modified bitumen roofs, single-ply roof coverings, and steep roof coverings.

6 Summary

Current roof life cycle data shows roof systems with effective service lives from about 10 to 25 years. Although there are many factors that influence life cycle, some of the more critical factors have been identified by research as heat aging, roof traffic, roof slope, and maintenance practices. Some of these factors have been shown to influence life cycle by reducing service life up to 25%. These factors should be considered in the design phase of new roof design, and in analyzing roof replacement alternatives. Clearly, more study is needed to analyze the effect of slope, maintenance practices, roof traffic, and the influence other factors on roof life cycle.

Regardless of the types of roof systems in the building inventory, few would argue against the cost savings features of an effective roof management system. Roof system management can be as simple and cost-effective as knowing the type, age, and area of each roof system on a campus or facility. This type of information is the least costly information to collect and maintain. It serves as the first step toward a more comprehensive and detailed management system.

The more detailed the inspection information and database management system, the more accurate the future maintenance and replacement cost predictions become. One of the benefits of a detailed roof management system is that an Owner can make better judgments as to which systems are working ***at their facility***. This leads to better decision making and longer service lives for each new or replacement roof system.

7 References

Paulson, Mari Ujka, Smith, John Z., Jr., (1998) The results are in. Published in Professional Roofing, April 1998. *National Roofing Contractors Association*, Rosemont, Illinois, USA, pp.14-18

Bailey, David M., Foltz, Stuart D., (1997) Performance of Polyvinyl Chloride (PVC) Roofing: Results of a Ten Year Field Study. *Fourth International Symposium on Roofing Technology*, Gaithersburg, Maryland, USA, pp.253-264

Kim, William A., Grecian, Donna S., Ketz, Richard J., Powell, Theresa R., Simmons, John A., (1997) On Investigating the Mechanism of Weathering of Chlorosulfonated Polyethylene Roofing and Exploring Maintenance Techniques. *Fourth International Symposium on Roofing Technology*, Gaithersburg, Maryland, USA, pp.37-49

Griffin, C.W., P.E., (1982) *The Manual of Built-up Roof Systems*. McGraw-Hill Book Company, New York, USA, pp.1-2

Cash, Carl G., (1997) The Relative Durability of Low-Slope Roofing. *Fourth International Symposium on Roofing Technology*, Gaithersburg, Maryland, USA, pp.119-124

Kyle, B. R., Kalinger, P., (1997) Service Life Prediction of Roof Systems by Reliability-Based Analysis. *Fourth International Symposium on Roofing Technology*, Gaithersburg, Maryland, USA, pp.50-57

Schneider, Kenneth G. Jr., Keenan, Anne S., (1997) A Documented Historical Performance of Roofing Assemblies in the United States 1975-1996. *Fourth International Symposium on Roofing Technology*, Gaithersburg, Maryland, USA, pp.132-137

Hoff, James L., (1997) Historical Warranty Repair Cost as a Measure of Long-term Roof System Performance. *Fourth International Symposium on Roofing Technology*, Gaithersburg, Maryland, USA, pp.125-131

114 DECISION-SUPPORT SYSTEM FOR SERVICE LIFE ASSET MANAGEMENT: THE BELCAM PROJECT

Z. Lounis,[1] D.J. Vanier,[1] M.A. Lacasse,[1] and B. Kyle[2]

[1]IRC, National Research Council Canada, Ottawa and [2]Public Works and Government Services Canada, Hull, QC, Canada

Abstract

This paper presents the framework of a stochastic decision-support system for life cycle asset maintenance management of building elements and systems. This system is based on a probabilistic modeling of the performance and risk of failure of building components. The maintenance optimization is formulated as a multi-stage and multi-objective optimization problem where the simultaneous satisfaction of several conflicting objectives, including minimization of maintenance costs, maximization of performance, and minimization of risk of failure, is sought. Compromise programming is used to determine the optimal ranking of the deteriorated components in terms of their priority for repair and replacement, by achieving a satisfactory trade-off between the competing or conflicting objectives. A product model of the building system is used to provide the data framework for collecting and processing data.

Keywords: Data collection, risk-based maintenance management, multi-objective optimization, product model, stochastic performance model.

1 Introduction

A review of recent Canadian construction statistics shows that $8.5 billion is spent annually on repairs and maintenance of buildings (Vanier 1998). An effective decision-support system is needed by asset managers to assist them in identifying optimal maintenance strategies that minimize life-cycle costs and risk of failure for a network of building systems and components. The development of such a system is the main objective of the *Building Envelope Life Cycle Asset Management*

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(BELCAM) project (Vanier and Lacasse 1996; Lounis *et al.* 1998). The BELCAM project has identified two achievable goals that will assist asset managers in the course of their work:

1. To develop reliability-based methodologies to predict the performance and service life of building components;
2. To establish protocols to optimize the maintenance management of building systems.

The use of the BELCAM system will enable facilities managers and owners to overcome some of the challenges facing them regarding: (i) prioritization of facilities for repair, and (ii) optimized time and repair action. The implementation of such a system will result in extended service lives for building components and systems, lowered life-cycle costs, and potentially lower risks of failure.

Given the time-dependence and uncertainty associated with roofing performance, Lounis *et al.* (1998) proposed a probabilistic Markovian model for the assessment and prediction of performance and probability of failure. The consequences of failure are evaluated from available data on maintenance and failure costs that depend on the building type, or occupancy, and mode of failure (i.e., water leakage, loss of energy control, loss of serviceability, deck collapse).

The maintenance optimization problem is formulated as a sequential, stochastic and multi-objective optimization problem; where the satisfaction of several conflicting objectives is considered, including: (i) minimization of maintenance and repair costs; (ii) maximization of network performance; and (iii) minimization of risk of failure. The multi-objective optimization problem is solved by using compromise programming methods (Lounis and Cohn 1995) that seek to determine the optimal ranking of deteriorated roof sections, in terms of their priority for repair and replacement, by achieving a satisfactory trade-off between all competing objectives.

The probabilistic performance prediction model and the multi-objective optimization procedure have considerable data integration and communication demands that will be effectively met through the use of a product model. Product modeling refers to the digital representation of elements that completely defines a product for all applications over its expected life. The proposed product model requires three major components (Vanier 1998): (i) elemental aggregation model of typical roofing systems; (ii) classification model of in-field roof performance characteristics; and (iii) instantiation of the subject roof portfolio.

This paper describes the framework needed to collect and analyze data required to prioritize repair and replacement projects in a given roof portfolio. It also identifies how the Markovian models, multi-objective optimization procedure and product models can be used to assist decision makers maximize the return on investment of their maintenance expenditures. The BELCAM project is concentrating on the life cycle maintenance management of low-slope roofs in the initial three years of the project. The methodologies developed during the course of the BELCAM research will be readily applicable to other building envelope systems.

The proposed decision-support system is shown schematically in Figure 1, and consists of the integration of the following three subsystems or enabling technologies:

1. Probabilistic models for performance prediction and risk assessment;
2. Multi-objective optimization procedure for decision making under conflicting objectives;
3. Product model of the roof system that provides the data framework for collecting
4. and processing data.

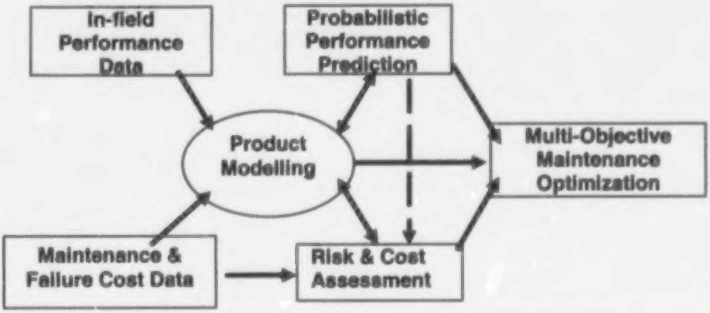

In-field Performance Data
Probabilistic Performance Prediction
Product Modelling
Multi-Objective Maintenance Optimization
Maintenance & Failure Cost Data
Risk & Cost Assessment

Fig. 1: Decision-support system for asset management: BELCAM methodology

2 Stochastic maintenance management system

The following section presents the framework for a stochastic maintenance management system that integrates a probabilistic performance prediction model and a risk assessment model with a multi-objective optimization algorithm.

2.1 Performance prediction using probabilistic Markovian models

The condition or performance of roofing components and systems deteriorate with time as a result of environmental degradation factors (temperature, solar radiation, water, wind), traffic loading, inadequate maintenance and poor workmanship. Given this fact, considerable uncertainty and variability are associated with the performance of roofing components, resulting from the uncertainty and variability in the environmental factors, quality of workmanship and maintenance level.

Therefore, a probabilistic Markovian model, and more specifically a discrete Markov chain that captures both the time-dependence and randomness of the roofing performance is used to model both the deterioration and repair processes. The condition of roofing components is represented by discrete condition ratings obtained by mapping the assessed damage levels obtained using the "*Roofer*" condition assessment system (Bailey *et al*. 1990) to the 1 through 7 rating scale shown in Figure 2.

The underlying assumption of the first-order Markov chain model is the dependence of the rate of deterioration on the current stress and cumulative damage only, and not on the entire stress history. The transition probability, p_{ij}, represents the likelihood that the roofing condition will change from state i at time t_n to a lower state j at time t_{n+1}.

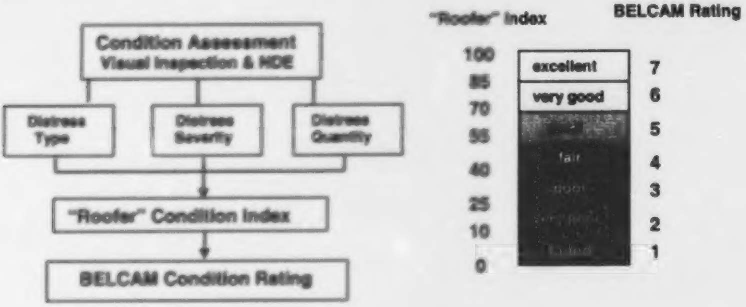

Condition Assessment
Visual Inspection & NDE
Distress Type
Distress Severity
Distress Quantity
"Roofer" Condition Index
BELCAM Condition Rating
"Roofer" Index
BELCAM Rating
100
85
70
55
40
25
10
0
excellent
very good
fair
poor
very poor
failed
7
6
5
4
3
2
1

Fig. 2: Condition assessment of roofing components

The development of the Markovian model requires a relatively limited amount of historical performance data at two or more points in time. Once the one-step transition probability matrix is generated, the future performance of the roofing component can be predicted using the n-step transition matrix as follows:

$$\mathbf{P}\{S(t_n)\} = \mathbf{P}\{S(0)\}\mathbf{P}^n \tag{1}$$

in which $\mathbf{P}\{S(t_n)\}$ is the state probability matrix at time t_n after n transitions; $\mathbf{P}\{S(0)\}$ is the initial state probability matrix; and $\mathbf{P}$ is the transition probability matrix. The probabilistic prediction of the performance using Eq. (1) is illustrated in Fig. 3, which indicates the evolution with time of the probability mass function of roofing performance.

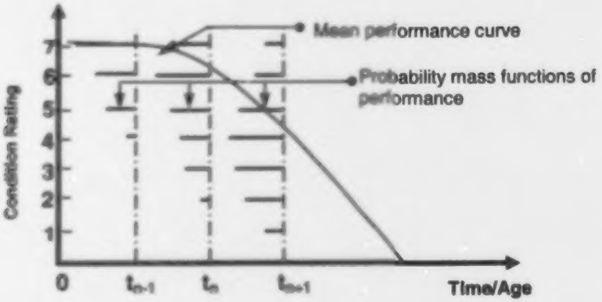


Fig. 3: Probabilistic Markovian modeling of deterioration

The transition probability matrix is determined from the historical performance data, collected during inspections, and is dependent upon several explanatory variables, including: age, environmental conditions, material type, quality of work executed as well as the amount and quality of maintenance.

2.2 Risk assessment

A modern built-up roof system has in general five basic components: namely, a waterproofing membrane, thermal insulation, flashings, structural deck and possibly a vapor or air barrier. In general, there is some correlation between the performance of different components, that in turn has a direct impact on the performance of the entire roofing system, and its risk of failure. A roof is a multi-component system with multiple failure modes that can be modeled as a hybrid system comprised of a combination of series and parallel subsystems. The probability of failure of each roofing component is time-variant and changes with time as a result of degradation or repair. The probability of failure can be determined using a systems-reliability approach, taking into account the correlation between different components and failure modes. In addition, the corresponding risk of failure of the roofing system may be evaluated once the consequences of failure are assessed. Two main failure modes can be identified:

1. Envelope failure, defined by the loss of the envelope main functions, namely water tightness and energy control;
2. Structural failure, or deck failure, that includes collapse and loss of serviceability.

The envelope failure at this time is the main concern of the BELCAM project. The cost of envelope failure depends on the value and vulnerability of the building contents under the roof, the costs of repair, energy costs and other incurred costs, such as costs of relocation and disruption.

2.3 Stochastic and multi-objective maintenance optimization system

At the network-level of roofing maintenance management, effective decision making requires the determination of the optimal prioritization of roofing projects for maintenance, repair and refurbishment (MR&R), including the time of repair and repair action. This optimization is a rather complex task, owing to: the large stock of deteriorated roofs; limited maintenance funds; uncertainty in the performance of roofing components; and, difference in the risk of failure associated with different buildings, roof sections and components. An effective maintenance management system should include all relevant objectives, such as: minimization of MR&R costs; maximization of network reliability; and minimization of risk of failure. Hence, it is clear that the actual maintenance management problem is multi-objective in nature and requires the determination of an optimal maintenance strategy to achieve the best trade-off between these conflicting objectives.

The multi-objective maintenance management problem can be mathematically stated as follows (Lounis and Cohn 1993; Lounis and Vanier 1998):

$$\min\ \mathbf{f}(x) = [\ f_1(x)\ \ f_2(x)\ \ldots\ldots\ldots\ f_m(x)\ ]^T \qquad x \in \boldsymbol{\Omega} \tag{2a}$$

$$\boldsymbol{\Omega} = \{x \in N\!: S_x(t)\ \leq 5\} \tag{2b}$$

where $\mathbf{f}$ = vector of objective functions; $\boldsymbol{\Omega}$= subset of the roofing network that at time t contains roofs with condition ratings (S_x) less than or equal to 5 and that require maintenance, and N= entire roofing network. The solution of this maintenance management problem can be obtained using vector optimization techniques, and more specifically, the compromise programming approach (Lounis and Cohn 1993, 1995; Lounis and Vanier 1998). In compromise programming, the "best" or satisficing solution is defined as one that minimizes the distance from the set of Pareto optima or efficient solutions to the so-called "ideal solution". This ideal solution is defined as the solution that yields the extreme (minimum or maximum) values for all objectives. The adopted priority index is based on the minimization of the normalized deviation from the "*ideal roof for repair*" that is associated with the "ideal vector objective $\mathbf{f}^*$" measured by the family of L_p metrics defined as follows (Lounis and Cohn 1995):

$$\min\ L_p(x) = \min_{x \in \Omega} \left[\sum_{i=1}^{m} \left| \frac{f_i(x) - \min\ f_i(x)}{\max\ f_i(x) - \min\ f_i(x)} \right|^p \right]^{1/p} \tag{3}$$

These L_p ($1 \leq p \leq \infty$) metrics indicate how close the satisficing solution is to the ideal solution. Another alternative to the above approach is possible through weighting of different objectives, depending on the sensibility of the decision maker toward risk (risk averse or risk prone), economics and network reliability.

The proposed optimization approach represents the first step towards the development of a comprehensive roofing management system. The next step is the optimization of MR&R strategies within a finite or short term planning horizon, followed by an optimization within a longer planning horizon at both network- and project- levels of management. This task is achieved through the combination of the dynamic programming approach with the proposed multi-objective optimization procedure

3 Regional surveys and data collection

The development of the probabilistic Markovian model is based on historical performance data of roofing components and systems collected throughout Canada. The participating Canadian agencies in the BELCAM project are identified by the star symbol in Figure 4. These sites represent a wide range of buildings in geographically diverse and hence, climatically different regions of the country. Each agency gathers information on roofs under their mandate and forwards the data for inclusion in a central database.

By gathering regional data, the project is generating performance profiles of sample roof sections that are representative of various climatic zones, construction techniques and maintenance practices and histories. Further data treatment will permit groupings relative to several explanatory variables. As additional performance data become available, the prediction model can be continuously improved using the Bayesian-updating method (Madsen *et al.* 1986).

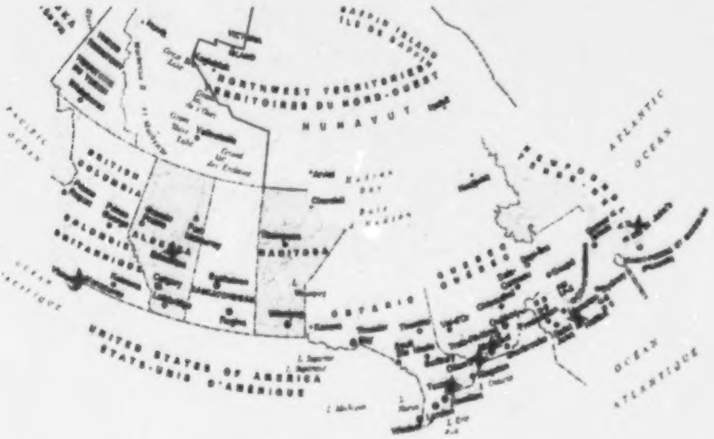


Fig. 4: Data collection sites and environmental characterization

3.1 Standardized data collection tools

Following an evaluation of available roofing inspection and maintenance management software (Vanier *et al.* 1998), *"MicroROOFER vers.1.3"* (Bailey *et al.* 1990; USACERL 1995) was chosen as the data acquisition software for the project. The software review examined four commercially-available packages relative to their ease of use, required minimum hardware configurations, operating platforms and database structure, their technical and reporting features as well as their ability to interface and link with other applications. MicroROOFER was considered to be the most comprehensive software package of the products studied, with respect to service life prediction.

The ROOFER methodology uses various quantified visual inspection indicators to determine a component condition rating for the roof flashing and membrane, whereas the condition rating of the insulation is determined from core sampling and evaluation of the percentage of wet insulation. These condition ratings conveniently map to the discrete condition states adopted in the discrete Markov chain model described earlier, and shown in Figure 2.

A review of existing data collection technologies lead to the expansion of the standard data collection framework to include the Fujitsu 1200 Stylistic™ pen-based computers running Microsoft Windows 95™ operating system. The utility of digital images was clearly recognized and, although not explicitly a component of the standardized data collection package, each agency participating in the regional survey is encouraged to use these visual recording techniques as part of their *"survey kit"*.

3.2 Roofing condition assessment information requirements

An examination of the existing data fields of MicroROOFER (USACERL 1995) and the BELCAM information requirements, relative to the identified explanatory variables, revealed numerous data gaps. In order to assure that adequate information was collected on all aspects of roofing performance (design and as-built conditions, material and workmanship quality, and condition of structural elements), additional requirements were mapped to appropriate storage locations within MicroROOFER. In all cases, the identified BELCAM protocol requirements can be recorded in existing "remarks" fields in MicroROOFER. The interaction between the explanatory variables, MicroROOFER data and BELCAM specific information is illustrated in Figure 5.

The Roofing Condition Assessment Survey (RCAS) methodology assists in obtaining consistent and easily interpreted information by detailing recommended methods of data collection and recording. A web-based *On-line RCAS Manual* provides a description of the MicroROOFER program requirements (USACERL 1995), as well as the data needed to fulfil requirements for the explanatory variable (Kyle 1998). An example of the recommended inspection procedures, rating scale and data input instructions for roof deck condition (as found in the on-line manual) is presented in Figure 6.

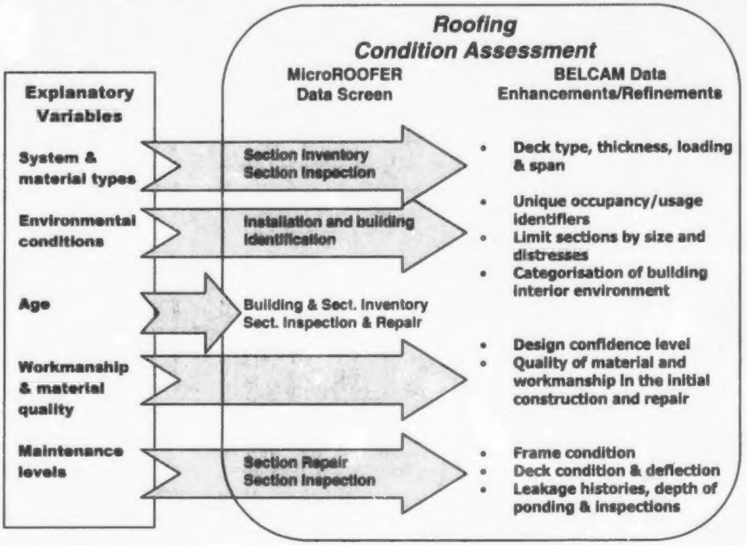
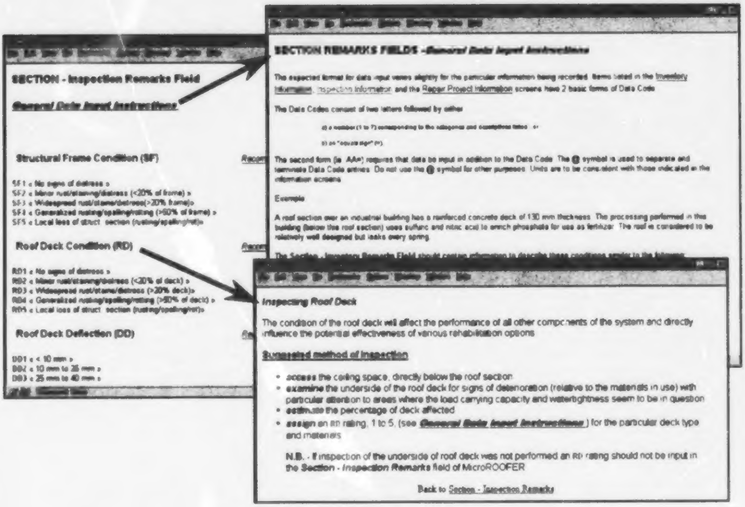

Roofing
Condition Assessment
MicroROOFER
Data Screen
BELCAM Data
Enhancements/Refinements
Explanatory
Variables
System &
material types
Environmental
conditions
Age
Workmanship
& material
quality
Maintenance
levels
Section Inventory
Section Inspection
Installation and building
identification
Building & Sect. Inventory
Sect. Inspection & Repair
Section Repair
Section Inspection
Deck type, thickness, loading
& span
Unique occupancy/usage
identifiers
Limit sections by size and
distresses
Categorisation of building
interior environment
Design confidence level
Quality of material and
workmanship in the initial
construction and repair
Frame condition
Deck condition & deflection
Leakage histories, depth of
ponding & inspections

Fig. 5: Explanatory variable data requirements for BELCAM



SECTION - Inspection Remarks Field

General Data Input Instructions

Structural Frame Condition (SF)

SF1 « No signs of distress »
SF2 « Minor rust/staining/distress (<20% of frame) »
SF3 « Widespread rust/stains/distress(>20% frame)»
SF4 « Generalized rusting/spalling/rotting (>50% of frame) »
SF5 « Local loss of struct. section (rusting/spalling/rot)»

Roof Deck Condition (RD)

RD1 « No signs of distress »
RD2 « Minor rust/staining/distress (<20% of deck) »
RD3 « Widespread rust/stains/distress (>20% deck)»
RD4 « Generalized rusting/spalling/rotting (>50% of deck) »
RD5 « Local loss of struct. section (rusting/spalling/rot)»

Roof Deck Deflection (DD)

DD1 « < 10 mm »
DD2 « 10 mm to 25 mm »
DD3 « 25 mm to 40 mm »

SECTION REMARKS FIELDS - *General Data Input Instructions*

The expected format for data input varies slightly for the particular information being recorded. Items listed in the Inventory Information, Inspection Information and the Repair Project Information screens have 2 basic forms of Data Code.

The Data Codes consist of two letters followed by either

a) a number (1 to 7) corresponding to the categories and descriptions listed ... or

b) an "equals sign" (=).

The second form (ie. AA=) requires that data be input in addition to the Data Code. The @ symbol is used to separate and terminate Data Code entries. Do not use the @ symbol for other purposes. Units are to be consistent with those indicated in the information screens.

Example

A roof section over an industrial building has a reinforced concrete deck of 130 mm thickness. The processing performed in this building (below the roof section) uses sulfuric and nitric acid to enrich phosphate for use as fertilizer. The roof is considered to be relatively well designed but leaks every spring.

Inspecting Roof Deck

The condition of the roof deck will affect the performance of all other components of the system and directly influence the potential effectiveness of various rehabilitation options.

Suggested method of inspection

- **access** the ceiling space, directly below the roof section
- **examine** the underside of the roof deck for signs of deterioration (relative to the materials in use) with particular attention to areas where the load carrying capacity and watertightness seem to be in question
- **estimate** the percentage of deck affected
- **assign** an RD rating, 1 to 5, (see ***General Data Input Instructions***) for the particular deck type and materials

N.B. - If inspection of the underside of roof deck was not performed an RD rating should not be input in the ***Section - Inspection Remarks*** field of MicroROOFER

Back to Section - Inspection Remarks

Fig. 6: RCAS instructions and rating scale for roof deck

The RCAS provides all regional surveyors with easily-accessed information on roofing defects, their definitions, how and what to inspect, as well as a consistent methodology to record specific roofing distresses. Use of the on-line RCAS manuals, coupled with the pen-based systems and digital cameras, reduces learning time required to conduct inspections and minimizes the time between inspection and data entry. Over 500 roofs are being surveyed in the course of the project, with inspections recurring annually.

4 Integration of decision-making tools

As mentioned earlier, the main requirements of the BELCAM project are: (i) collection of in-field performance data (as described above); and (ii) development of performance and service life prediction models using these data. The improved performance prediction model will lead directly to the optimization of maintenance expenditures. The following steps are needed to accomplish these requirements:

1. Regional survey crews collect baseline building information, including drawings
2. Crews collect electronic information on current condition using MicroROOFER, on-line RCAS Manuals and pen-based systems, including digital images of distresses;
3. Crews collect past maintenance information for individual roofs regarding repair

actions and associated expenditures to assess life cycle costs and MR&R investment level;
4. New data are uploaded to a central server to update the probabilistic model and the MR&R and failure costs;
5. Operator queries the central database on issues regarding specific roofing components to establish the remaining service life; and
6. The performance and service life data calculated above, in conjunction with risk and MR&R costs data, are used by the asset managers to optimize their maintenance expenditures using multi-objective and dynamic programming approaches.

5 Envisioned user interface

The decision-making tool must have an intuitive interface such as the one proposed in Figure 7. This interface would permit the user with quick and efficient:

1. Views of the current "*condition or performance profile*" of the roofing network and simulate its "change with time" as a result of degradation or maintenance;
2. Views of the current "*risk profile*" of the roofing network and simulate its "change with time" as a result of degradation or maintenance;
3. Views of the current "*maintenance cost profile*" of the roofing network and simulate its "change with time" as a result of degradation or maintenance; and
4. Sensitivity analysis by changing the weighting factors applied to the different objectives depending on the decision maker's risk aversion.

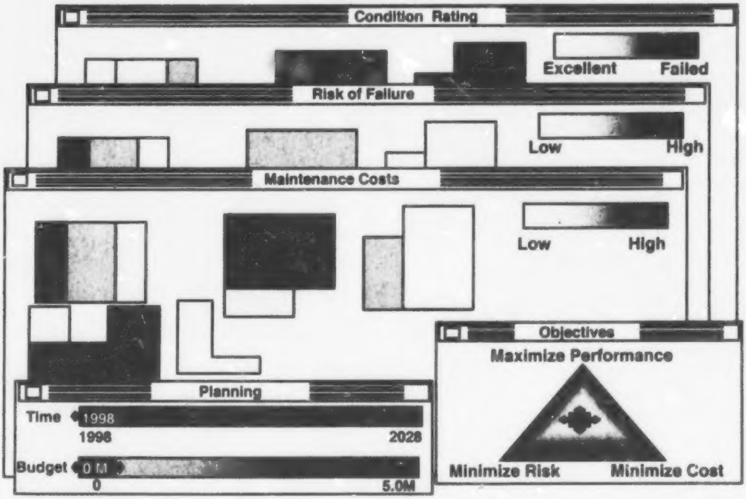

Condition Rating
Excellent
Failed
Risk of Failure
Low
High
Maintenance Costs
Low
High
Objectives
Maximize Performance
Minimize Risk
Minimize Cost
Planning
Time
1998
1998
2028
Budget
0 M
0
5.0M

Fig. 7: Proposed user interface

6 Conclusions

A stochastic and multi-objective decision-support system is proposed for the optimization of roofing maintenance management at both network and project levels. A discrete Markov chain model is used to predict the performance of the roofing system during its service life. The maintenance optimization aims at simultaneously satisfying several conflicting objectives, including the minimization of the MR&R costs and risk of failure and the maximization of the roofing network reliability or condition rating. Compromise programming is used to determine the optimal ranking of deteriorated roofs in terms of their priority for repair and replacement.

BELCAM's first goal of developing techniques for service life prediction is addressed by the probabilistic Markovian modeling of performance of roofing components. The second goal of optimizing maintenance management is addressed by multi-objective optimization. These two goals could not be addressed without the use of product modeling, which provides the data integration requirements for this project. The research plan is presented in this paper; however, considerable work is still required to collect the requisite data, to integrate the required applications, and to test and validate the techniques proposed in this paper. The decision-making tools proposed in this paper will assist asset managers to predict the remaining service life of roofing systems and will allow them to optimize the expenditures of their maintenance dollars. Conceptually, the techniques developed in the BELCAM project are readily applicable to other building envelope systems.

7 References

Bailey, D.M, Brotherson, D.E., Tobiasson, W., and Knehans, A. (1990), "ROOFER: an engineered management system (EMS) for bituminous built-up roofs", *USACERL Technical Report M 90/04*.

Kyle, B.R. (1998), "*Roofing Condition Assessment Survey (RCAS) methodology within the BELCAM framework: HTML on-line manual*", National Research Council, Ottawa, Canada.

Lounis, Z., and Vanier, D.J. (1998), "Optimization of bridge maintenance management using Markovian models", *Proc. 5th International Conference on Short and Medium Span Bridges*, Calgary, Vol. 2, pp. 1045-1053.

Lounis, Z., Vanier, D.J., and Lacasse, M.A. (1998), "A discrete stochastic model for performance prediction of roofing systems", *Proc. CIB World Congress*, Gävle, Sweden, Vol. 1, pp. 305-313.

Lounis, Z., and Cohn, M.Z. (1993), "Multiobjective optimization of prestressed concrete structures", *ASCE J. of Struct. Engrg.*, Vol.119, No.3, pp. 794-808.

Lounis, Z., and Cohn, M.Z. (1995), "An engineering approach to multicriteria optimization of highway bridges", *J. of Microcomputers in Civ. Engrg.*, Vol.10, No.4, pp. 233-238.

Madsen, H. O., Krenk, S. and Lind, N.C. (1986), *Methods of Structural Safety*, Prentice Hall Inc., Englewood Cliffs, N.J.

Ross, S.M. (1996) *Stochastic Processes*, 2nd Ed., John Wiley & Sons, New York.

USACERL (1995). *MicroROOFER*, Version 1.3. U.S. Army Corps of Engineers, Construction Engineering Research Laboratories.

Vanier, D.J. and Lacasse, M.A. (1996), "BELCAM project: service life, durability, and asset management research", *7th Conf. on Durability of Building Materials and Components*, Stockholm, Sweden, Vol. 2, pp. 848-856.

Vanier D.J. (1998), "Product modeling: helping life cycle analysis of roofing systems", *The life cycle of construction it innovations: Technology transfer from research to practice*, Stockholm, Sweden, pp. 423-435.

Vanier, D.J., Doshi H., Kyle, B.R., and Marcellus, R.W. (1998) "Roofing maintenance software review: The art of roofing condition inspections." *Roofing Contractors Institute Interface*, Vol. XVI, No. 3, pp. 10-18.

115 CONDITION ASSESSMENT OF FLAT ROOFS, INCLUDING THE USE OF EXPERT SYSTEMS

G.K. Saunders and A. Goodier

Building Research Establishment Ltd., Watford, UK

Abstract

This paper reports the current state of condition assessment flat roofing and its impact on assessing the durability of flat roofs in the UK. The paper is intended to provide information to support the development of approaches to assessing the durability of flat roofing. This included the development of expert system and to this end the paper report work undertaken by or in close co-operation with the BRE upon diagnosis and condition assessment expert systems.

Keywords: flat roofing, condition assessment, expert systems.

1 Introduction

The typical design life of a building in the UK is considered to be 60 years. Recent experience has shown that the durability of flat roofs is not that long and the roof will need to be replaced at least once during the building life span. In effect the 60 years life is a notional lifetime attributed for insurance purposes and buildings do last longer than 60 years. The waterproof membrane of a flat roof will need replacing at some stage during the service life of the building due to the durability of the membrane affected mainly by natural weathering. In the U.K. there is a substantial amount of information in the BFRC/CIRIA guide on "Design and Good Practice". It recognizes that the relatively recent inclusion of thermal insulation together with changes in building methods has led to the premature failure of a number of flat roof membranes. The reason for these failures can be due to design, materials or their interaction and installation. A failure of the roof membrane can be a combination of these factors resulting in sometimes a unique failure exceptional to one particular

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roof, this can make assessing the durability at the design stage very difficult. There are however a number of faults which do occur frequently and can be recognized so that appropriate guidance can be given to avoid them in the future. Recognizing these faults and their symptoms is a part of the information given by a condition survey of the roof. Being able to assess the importance and impact of these faults is an important step in developing an approach to predict the durability of flat roofing.

The next section of this paper details the current state of the art in terms of flat roofing and the assessment of flat roofing in the UK. It is strongly felt that this information will be a critical factor in developing a successful approach to assessing and predicting flat roof durability.

1.1 Materials used in the UK for flat roofing membranes

The majority of flat roofs in the UK have membranes made from bituminous materials. The use of plastic or rubber sheets for flat roof waterproofing has increased in recent years.

1.1.1 Bitumen roofing felts

In the UK it is common for a bituminous membrane to be built-up in a number of layers. The roofing felt is supplied as a reinforcement or carrier that has been impregnated and coated with bitumen as part of a manufacturing process. The surface is coated with fine sand to prevent sticking in the roll. It is common for capsheets to be coated with mineral flakes or granules. The reinforcement or carrier can be organic fibre, asbestos, mineral (including glass) or polyester. Although asbestos carriers are no longer manufactured in the UK it is still possible to find roofs which have this type of covering. The most common way of forming a built-up membrane is by fusing the layers together using hot molten bitumen.

Although the bitumen can be polymer modified a built-up membrane which has polyester reinforcements is referred to as "high performance" regardless of the type of bitumen used. Built-up membranes made from high performance felts have been used successfully in the UK and have a service life of at least 20 years.

1.1.2 Mastic asphalt

Mastic asphalt is a bituminous material that has fillers such as limestone added to it together with fine aggregates to form a hot liquid applied membrane which can be used for roofs, floors or roads. In the UK mastic asphalt has been widely used for the waterproof membrane on concrete decks. Mastic asphalt is laid over a separating membrane to isolate it from movements in the substrate. There are examples of mastic asphalt roofs about 70 years old. The introduction of thermal insulation into roof designs and the widespread use of lightweight decks has shown that traditional mastic asphalts do not tolerate movement especially at the edges of the roof. Modern asphalt is modified with suitable polymers to enhance its performance.

1.2 General condition survey of a flat roof

This section deals with a general condition survey developed at the Building Research Establishment to be included in a Good Repair Guide (BRE Good Repair Guide 16). It shows how much information is required just to interpret a simple external inspection. There is insufficient space here to show what factors influence the next important decision to replace or refurbish based on the condition survey.

Before embarking on any work on a flat roof it is essential to carry out a careful and detailed assessment of its condition. There is no point in carrying out repairs to the waterproofing membrane if there are serious problems in the roof structure. It is also wasteful to replace an otherwise sound roof when a repair would have been adequate.

1.2.1 History of the roof

The first thing to do is to find out about the roofs history. The information needed is;

- How old is the covering
- Has the covering been repaired before
- For how long has the roof been defective
- If the roof is leaking, does it leak during or immediately after rainfall? or a few days later?
- Does water pond on the roof?
- What is the room below used for?' (Flat roofs above kitchens, bathrooms or utility rooms are more vulnerable to condensation problems than those above living rooms or garages.)
- Are there plans for change of use? is the roof used as a balcony or terrace? or are there plans to use it as such.

1.2.2 Visual inspection

The next step is to make a thorough visual inspection, both outside and inside the building. Before getting up on the roof make sure it is safe to do so. If the roof is sagging between joists do not walk on it. If possible carry out an inspection immediately after rainfall so that you can see any ponding or leaks and watch the speed of drainage.

1.2.3 Inspection check list

Table 1 below is a check list for an external inspection together with an interpretation of what the findings mean. This is general advice and is based on many years of inspecting flat roofs.

1.2.4 Other considerations

The BRE Good Repair Guide also has a check list for looking at the edges of the roof where faults often occur at edge details. Detailed information is given on faults in membranes, checking for condensation, problems with parapet walls, thermal insulation and fire performance.

Table 1: Typical inspection check list

What to look for in the general flat area of a roof.	Cause or Fault
Are there any signs of previous repairs, and are they all in the same place?	This indicates that the original repair has been done poorley. Carry out proper new repair. Several patches mean a general problem. If the roof has dried out then overlay with a new covering. Otherwise, consider replacement.
Has the membrane been punctured? eg by cable clips.	If yes, a localised patch repair is needed. If there is a line of holes then it may be better to overlay with a strip of high performance felt.
Is there flowing, rippling, cracking or crazing?	This tends to indicate old age, particularly of mastic asphalt.
Are there splits holes or ruckles ?	A hole may be caused by damage. Localised patch repair will be sufficient if cause can be established. Ruckles and splits are associated with substrate movement.
Are there blisters?	The likely cause is entrapped water or condensation. If blisters are small, inspect regularly. If they grow then further investigation of a condensation problem is needed.
Have the lap joints pulled or lifted?	This can be a problem with older membranes. In severe cases remove the old covering and replace with high performance felt checking other roof components. Filling with sealant is a short term solution and is rarely effective.
Has the roof been surfaced with chippings, solar-reflective paint or a factory-applied mineral surface(bitumen felts only)? Is the surface damaged or absent.	Solar reflective paints need maintenance about every 5 years. The wrong types on mastic asphalt can induce surface cracking. Chippings may be blown away by the wind if they are not secured in a surface dressing. In this case, the old ones should be removed and a new dressing compound and chippings applied.
Is there ponding of surface water?	Ponding can be tolerated by high performance felts and mastic asphalt. Combined with other symptons it can indicate major problems.
• Is there excessive deflection, springiness or sponginess in the roof.	Deflection or sagging may affect the flow of water off the roof resulting in ponding and additional loading on the structure. It may be caused by : incorrectly designed structural components, or degradation of components
• Is there flooding in gutters or above roof outlets.	This indicates a drainage problem. If flooding extends up to flashing level water can enter the building.

1.3 Repair, refurbish or replace

The decision to repair or replace the waterproof membrane is not an easy one to make despite having carried out a detailed assessment of the roof. Cost is always an important consideration but there does come a point when patching an old roof frequently is less cost effective than replacement with a new roof, especially considering the internal damage a severe leak may cause. A table has been developed in the Good Repair Guide which gives further guidance on the decision to repair or replace taking into account age of the roof. It is also apparent that a roof may exhibit more than one sympton and that the whole table should be studied before making a decision.

1.4 Comments on roof survey

The checklist stated above is just a part of that required for a full and comprehensive roof survey. The results of the condition survey lead to many situations and the BRE Good Repair Guide is only general in nature. To cope with the situation that many roofs are unique would require the development of a much larger set of rules and causes which in paper form would be impractical to develop.

2 Introduction to the use of advanced information technology in support of flat roof condition assessment

The earlier parts of this paper have explained the current state of the art in flat roofing assessment. The BRE Good Repair Guide recognises that observations made at different intervals during the life of a roof covering will lead to different repair proposals. The current view is that a general estimate of service life is 20 years. More information of a specific nature is required to predict service life which in turn means that a much larger set of rules are needed. Other means than note pad and paper are required to gather and interpret this information for a wide range of systems. This part of the paper therefore explains the advances in the use of information technology in support of flat roofing assessment, using as an example a system called ROOF which was developed to aid the diagnosis of flat roofing defects.

A number of technologies collectively known as expert systems have been explored by the BRE. Expert systems are an information technology technique that falls within the IT area known as Artificial Intelligence (AI). Formally AI is the use of techniques developed by the AI community with the aim of making computers reason like human beings. The use of expert systems within the building industry does not have this aim, but is focused on the application of AI techniques to aid the development of computing systems to aid engineering tasks. The use of the term expert system is a misnomer for the role of such computing systems. Expert systems are not experts in a field of knowledge, at best the level of intelligence displayed could possibly be described as "a steady plodder". Expert systems do not generally invent new appreciation of knowledge or understanding of a new problem, but are very good a applying knowledge it already has, without omitting or overlooking some part of the knowledge or task related to the problem being examined. The authors prefer the term "Advisory" or "Assistant" for such systems rather than "Expert".

One significant advantage that the BRE sees deriving from the extensive use of expert systems is the formalisation of the data collected from site visits and examinations. A significant problem for the general assessment of building stock (for diagnosis, condition assessment or service life prediction) is the lack of data. What data that has been collected is extremely variable from one assessment to another assessment. This makes any formal (e.g. statistical) analysis of detailed aspects of the buildings behaviour difficult. To overcome this difficulty the data collection will need to be standardised, this can be best achieved by using an expert system to guide the data collection.

The work on ROOF is only one of several expert systems [Allwood et al 1986; Goodier and Matthews 1996; Dutton, and Maun 1996] that have been developed by or in close co-operation with the BRE. ROOF is a large rule based expert system that can diagnose over 270 defects affecting flat roofs. This diagnosis is undertaken using a rule base of over 1000 rules.

2.1 Background to the development of a roofing diagnosis system

There are numerous reasons why it is advantageous to develop such a computer system to assist the diagnosis of defects in flat roofing, these include:

1. Retention of expert knowledge. The development of an individual into an expert upon a subject is a very expensive task, and is generally achieved over a long period of time. When the individual is lost to the organisation (typically through retirement) the individuals knowledge is also lost to the organisation. A system incorporating some of the individuals knowledge is an effective way of mitigating some of this loss.
2. De-skilling of the task. As mentioned above experts are very expensive, if information about a defective roof can be collected by a less skilled individual supported by an expert system this can be extremely beneficial to the organisation.
3. Ensuring rigourous data collection. Regardless of the quality of the individual undertaking an assessment, important information, data and observations can be easily overlooked. Expert systems can help to avoid this by guiding the information collection. However, importantly, this guidance is targeted and refined and does not suffer from the restrictions of a checklist.
4. Support for design and specification. The ROOF system was designed not only to support diagnosis, that is determining the cause of a roof defect but to also support condition assessment and design checking.

A number of programming techniques were considered, these included:

1. Traditional Procedural Programming. Traditional programming such as 'C' or HTML/Java (web pages) has a number of advantages, such as, a fixed path from start to finish and a inexpensive development environment. However encoding interactive knowledge within such systems is extremely difficult and requires a clear understanding of the knowledge paths and often leads to considerable duplication.
2. Case Based Systems. Case Based Reasoning systems are an expert system technique where example problem cases (with known solutions) are identified as being close or similar to the problem (case) being considered. However the use of this technique for diagnosis (and other tasks) in buildings is restricted due to the lack of readily available and formatted example cases.
3. Rule Based Systems. Rule based systems are another expert system technique. This technique encodes the knowledge (e.g. the knowledge about roof diagnosis) in the form of IF <condition > THEN <conclusion> rules. The set of rules encoding the knowledge is known as a Rule Base. This rule base is searched using a software tool known as an inferencing engine to find the solution(s) to problems. This was the expert system technology adopted for ROOF.

It was envisaged that the ROOF system could be used at several points in the life cycle of a roof, these include:

1. The primary role of ROOF was to aid the on site diagnosis of defect of a flat roof. This can be used to aid the estimation of service life. When the cause (and hence seriousness) of the defect is correctly identified with a significant degree of confidence then the decision about repair or replacement can be undertaken with an improved degree of confidence.
2. As a tool to check condition. It is obvious that the knowledge used by an expert to diagnose a defect in a roof has a lot in common with the knowledge used to check a roof condition and likely service life. The use of the knowledge collected and encoded for diagnosis in ROOF for condition assessment was examined in some detail (Allwood and Goodier 1995c)
3. As a tool to check designs. The checking of a design is very similar to condition assessment except that the roof only exists on paper.

2.2 The development of the ROOF system

The next few sections of this paper detail the development stages for the ROOF system. This careful staged development is arguably more important for expert systems than for traditional programming. The stages here are similar to those followed for the development of other expert systems at the BRE (Goodier and Matthews 1996) and form a useful guide to follow when developing similar systems either for diagnosis, condition assessment or service life prediction. The descriptions of the stages presented here are brief, more detail can be found in other publications (Allwood and Goodier 1995a, 1995b). The stages in the development of the roof systems were:

1. The development of a specification.
2. Producing a list of goals for the system to identify.
3. Collecting the knowledge used to establish the goals.
4. Encoding the knowledge.
5. Testing the system.

2.2.1 Developing the specification

The development of the specification for ROOF had to consider a number of factors, these included:

1. The target user of the system. What level of expertise would the user have, this could vary from an experience roof inspector that was using the system as a supportive tool to a individual with no technical experience (the caretaker). The target user for ROOF was set as a building professional but with no experience of roofing.
2. The hardware system. The system was designed to run on a laptop computer. It is easily capable of running on a compact laptop such as a Toshiba Libretto.
3. The operating system. At the time of the initial development of ROOF MS Windows was not an established environment for technical computing. However it was decided to develop ROOF in this environment, this proved to be a fortuitous choice.
4. The development system. A detailed examination of the expert system shells (the

software systems used to develop and run rule-based systems) was undertaken to establish the best shell.

5. The degree of help to be given. The help to be included in an expert system has to be balanced between the competing requirements of clarity and completeness.

2.2.2 Producing a list of goals for the system to identify

A Goal is the expert system terminology for the fundamental cause of a defect that can affect a roof. When undertaking a survey it is one or more of these goals that the system is attempting to identify. It is the identification of these goals that is the first technical stage in the development of any rule-based system. In the building industry the identification of these goals is not trivial. Many experts do not find it easy to list goals as they represent the end result of their thinking whereas a consultation will start with symptoms. This requires that the system developers develop techniques to aid the identification of the goals [Allwood and Goodier 1995b]. The development of the goal list for ROOF was no exception to this. The goal identification exercise established 276 fundamental causes of defects that could be found on a flat roof.

2.2.3 Collecting the knowledge to identify goals

Once the set of goals have been established the main work in the development of the expert system can be undertaken, that is, the collection of the knowledge to identify and establish the goals. This process is known as knowledge elicitation. Knowledge that is used to establish goals can be collected from several sources, such as; published literature, example cases of defective roofs and directly from experts. All of these were used in the development of ROOF, but the most significant was the knowledge collected directly from experts. The knowledge was collected from the experts in a series of interviews. The interviews had a range of formats, such as informal discussions about roof diagnosis, review of example investigations, to, structured interviews to collect specific knowledge. The knowledge elicitation for ROOF was aided by the recognition of structure within the knowledge [Allwood and Goodier 1993, 1995b]. If structure can be found, this allows the elicitation tasks to be broken down, the knowledge elicitation is then considerably eased and the accuracy significantly improved.

2.2.4 Encoding the knowledge

One of the significant advantages of rule based expert system is that encoding the knowledge (if it has been collected in a structured manner) is relatively easy, though still not a trivial task. Figure 1 shows an example rule from the ROOF system. Figure 1 shows the benefit of establishing a structure to the knowledge that can be reflected in the encoded rules. Each set of rules that can be used to establish the cause of a defect has two halves. One half establishes that the symptoms on the roof are such that the cause of the defect can occur. The second half checks that the symptoms confirming the cause of the defect exists. The encoding of the knowledge was further aided by recognising the existence of meta knowledge (knowledge about knowledge) and how this influenced the system (Allwood and Goodier 1993). In rule based systems knowledge is not only encoded in the rules, but also exists in the questions and help screens, this is the case with ROOF.

```
if situation allows cause c3.5
and symptoms indicate cause c3.5
then causes include c3.5 built up roofing
improperly spanning expansion joint

if type of membrane is built-up roofing
and built-up membrane had base of low grade
and roof deck includes movement joint
and membrane does not include movement detail
then situation allows cause c3.5

if presence of splits is yes
and orientation of splits include in-line with movement joint
and location of splits include at movement joint

or offset slightly from movement joint
then symptoms indicate cause c3.5
```

Fig. 1: Example rules from the ROOF system

2.2.5 Checking the system

Checking expert systems is not an easy task, primarily because the route to a conclusion is not fixed. ROOF was checked in several ways. One advantage of rule based systems is that the individual rules are English like and can be read by an expert. This was undertaken for ROOF. The system was taken on a number of site visits and tested against roofs with defects where the cause of the defects were known, and its' performance checked. ROOF was further tested using a technique known as blind testing. Here the data from a number of case studies were entered into the system and presented to experts. The solution found by both the systems and the experts were anonymously presented to an adjudicating expert. This enabled the systems' performance to be compared to an expert. The system performed very well in these tests.

2.3 Future expert systems development at the BRE

The BRE remains heavily involved in the development of expert systems for diagnosis, monitoring and specification. These systems can be expected to contribute to service life prediction. Two areas are currently underway. The first is the development of a real time expert system to aid structural monitoring, this follows on from earlier work [Goodier and Matthews 1996]. The second is the development of an expert system to aid flat roof specification. Real time expert systems to aid structural monitoring perform several functions such as improving the presentation of information, advanced sensor validation and applying engineering knowledge to check the structure. In many situations monitoring is the first stage in condition assessment, an important part of service life prediction. Benefits from the improved performance of monitoring can be expected to cascade down. The second system to aid flat roof specification will be based on standards, manufacturer's data and published performance figures to make design less laborious and more accessible.

3 References

Allwood, R.J., Shaw, M.R. *et al.*, (1986) Building dampness diagnosing the causes, *Building Research Practice*, Vol. 16, No. 1, pp37-42.

Allwood, R.J., Goodier, A., (1993) Overall strategies of diagnostic expert systems. In *Information Technology for Civil and Structural Engineers*, (ed. B.H. Topping & A.I Khan .), A.I. Civil-Comp Press, Edinburgh, pp73-78.

Allwood, R.J., Goodier, A., (1995a) A knowledge based advisor to diagnose defect in flat roofs. *Chartered Surveyor Monthly*, Vol. 4, No. 6, pp32-33.

Allwood, R.J., Goodier, A., (1995b) Development of a practical knowledge based adviser to diagnose defects in flat roofs. *ASCE Second Congress on Computing in Civil Engineering*, Atlanta, GA, Vol. 1,pp365-372

Allwood, R.J., Goodier, A., (1995c) Structuring diagnostic knowledge for multiple usage. *Advance in Engineering Software*, Vol. 26, pp19-22.

Building Research Establishment (1998) Good Repair Guide 16. Part 1. Flat Roofs: assessing bitumen felt and mastic asphalt roofs for repair. *Construction Research Communications Ltd. 151 Rosebery Avenue, London EC1R 4QX.*

The British Flat Roofing Council and Construction Industry Research and Information Association (1993) *Flat Roofing: Design and Good Practice. CIRIA. 6 Storey's Gate, Westminster, London, SW1P 3AU*

Dutton, D. and Maun, K. (1996), CBR for the Selection of Appropriate Timber. In *Design, Proceedings - 2nd UK Workshop on CBR*, (Ed. I. Watson), University of Salford, Salford, UK, pp. 97-105.

Goodier, A., Matthews, S.L., (1996) Knowledge based systems applied to real-time structural monitoring. In *Information Processing in Civil and Structural Engineering Design* (ed. B. Kumar), Civil Comp Press, Edinburgh, pp263-270

PART EIGHT

PROBABILISTIC AND RELIABILITY METHODS

116 A STOCHASTIC APPROACH TO THE FACTOR METHOD FOR ESTIMATING SERVICE LIFE

L.I. Aarseth and P.J. Hovde

NTNU, Trondheim, Norway

Abstract

The ISO/CD 15686-1 "Service life planning" describes a deterministic method that allows an estimate of the service life to be made for a particular component or assembly in specific conditions. In "real life" the service life has a big scatter and should be treated as a stochastic quantity. In this paper we introduce the "step-by-step" principle as a stochastic approach to the ISO factor method. The "step-by-step" principle provides a more systematic approach to the estimating process and makes possible a stochastic handling of the factors. For each factor three estimates shall be made; the minimum, maximum and the most expected value of the factor. In this way the uncertainty is identified and estimated for each factor. The most uncertain factors should, if possible, be divided into sub-elements and more information should be gathered in order to reduce the uncertainty. In this stochastic approach the "factors" are treated as elements which finally are summed up. Also unlike the proposed ISO factor method, the estimates are expressed in years, instead of in numbers close to 1. These changes facilitate seeing the consequences of the estimates during the estimating process. After a statistical calculation the estimated service life is expressed as three figures; the expected value plus/minus one standard deviation. Two examples are shown where the service life is estimated for a window: first in a deterministic way according to the proposed ISO factor method, then in a stochastic way according to the proposed "step-by-step" principle.

Keywords: Estimating service life, "step-by-step" principle, stochastic approach, ISO factor method

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1 Introduction

The ISO/CD 15686-1 "Service life planning" describes a method that makes possible to estimate the service life of a particular component or assembly under specific conditions. An important shortcoming in this method is that the service life is treated as a deterministic value. In reality the service life has a big scatter and should be treated as a stochastic quantity (Siemes 1997). A stochastic treatment requires the aid of a statistical tool, and preferably a tool designed for practical utilisation. In this report the "step-by-step" principle is introduced as a stochastic approach to the ISO factor method.

2 The ISO factor method

The following definitions are taken from "ISO 6707 Vocabulary of construction terms – Part 1 General terms" (as referred in the ISO/CD 15686 – 1; 1997).

Table 1: Definitions

Service life (SL)	Period of time after installation during which all essential properties of an item meet or exceed the required performance.
Design life (DL)	Period of use intended by the design, e.g. as established by agreement between the client and the designer to support specification decisions.
Reference service life (RSL)	Service life established for a class of building or for parts of a building for use as a basis for estimating service life in specific items in specific conditions.
Estimated service life (ESL)	Reference service life multiplied by factors related to specific conditions, e.g. materials, design, environment, use and maintenance

The method allows an estimate of the service life to be made for a particular component or assembly in specific conditions. The method is based on a reference service life and a series of modifying factors that relate to the specific condition of the case. The reference service life is the expected service life in the conditions that generally apply to that type of component or assembly.

Table 2: Deterioration factors of materials and components

Factors		*Relevant Conditions (examples)*	
Agent related to the inherent quality characteristics	A	Quality of components	Manufacture, storage, transport, materials, protective coatings (factory-applied)
	B	Design Level	Incorporation, sheltering by rest of structure
	C	Work execution level	Site management, level of workmanship, climatic conditions during the work execution
Environment	D_1	Indoor environment	Aggressiveness of environment, ventilation, condensation
	D_2	Outdoor environment	Elevation of the building, micro-environment conditions, traffic emissions, weathering factors
Operating Conditions	E	In use conditions	Mechanical impact, category of users, wear and tear
	F	Maintenance level	Quality and frequency of maintenance, accessibility for maintenance

Any one (or any combination) of these variables can affect the service life. The factor method can therefore be expressed as a formula in which:

ESLC = RSLC x A x B x C x D_1 x D_2 x E x F"

The output of the equation is a single number; the estimated service life of the component (ESLC). The following quotation from chapter 7 in the ISO/CD 15686 – 1 indicates the intended use level of the factor method (in 1997):

> *"The factor method is a way of bringing together the consideration of each of the variables that is likely to affect service life. It can be used to make a systematic assessment even when little or no reliable test data is available. Its use can bring together the experience of designers, observations, intentions of managers and manufacturers assurances as well as some data from test houses.*
>
> *The factor method does not provide an assurance of a service life - it merely gives an estimate based on what information is available. It is less reliable than a fully developed prediction of service life. The distinction between estimated and a predicted service life should be made when a forecast of service life is given. The information taken into account should also be recorded, so that it is clear whether the estimate is particularly robust or not.*
>
> *It may be desirable to consider the consequences of failure when estimating service lives using the factor method. It may be used as a guide both to those components which should be included in estimates (it is not necessary to estimate every component's service life) and in deciding those where failure would be most critical. For these it may be necessary to be extra cautious, either by raising the design life (the "target") or by being particularly rigorous in assessing the value of factors.*
>
> *The factor method can be applied to both components and assemblies. When applied to assemblies it is necessary to consider the interfaces (e.g. joints) between components as well as the components themselves. For example, different external environment and maintenance factors may apply to a whole assembly that relies on sealants to weatherproof the joints between factory made cladding units than that which would apply to the each of the individual cladding units."*

3 The "step-by-step principle"

The conceptual context in the original description of the "step-by-step" principle is project planning. In this paper most notions are left unchanged but, where necessary, their relevance for estimating service life is emphasised. The "step-by-step" principle is based on "*Successivprincippet*", which was developed by Lichtenberg (1990) at DTU in Denmark in the early 70's. The "step-by-step" principle was developed at NTNU in Norway (Klakegg 1993) as a tool for improving the quality of the basis for decisions in project planning under uncertainty. As with estimating service life, a basic problem in project planning at an early stage is the lack of relevant information.

In "The step-by-step principle – a systematic approach to project planning under uncertainty" Klakegg (1993) states that the "step-by-step" principle, when applied to e.g cost planning, puts focus on the way the plans are made, and not only on the contents of the plans. Improved communication and awareness of the process of planning is vitally important in order to improve the plan itself. This is why the "step-by-step" principle is applied in the planning session with a resource group, consisting of planners and project members, guided by a moderator. The "step-by-step" principle is built on knowledge from professions like psychology, sociology, pedagogy, creative problem solving, and management formed into a consistent and flexible working procedure. This procedure offers support to the process of documenting the basis for decisions in accordance with the modern philosophy of quality management. The aim of implementing a systematic approach to planning under uncertainty is normally to improve the economic benefit of the project. This is achieved by improving the basis for decision-making and improving control and management in the project by focusing on the situation in which decisions are made. Practical use of the "step-by-step" principle has demonstrated that the plan is realistic and relevant for decision-makers.

In traditional project planning, estimates of costs, time consumption etc. are handled as deterministic values. The uncertainty of these estimates is taken into account after the main calculations are done, and then added to this sum as a "safety value". In the "step-by-step" principle, the uncertainty is identified and estimated for each element (cost, time etc.) during the calculation. Thus the "step-by-step" principle identifies the most uncertain elements. These elements may then, if possible, be divided into sub-elements and/or more information can be gathered in order to reduce the uncertainty.

3.1 The main steps in the "step-by-step" principle

Klakegg (1994) describe the main steps in the "step-by-step" principle as shown in Figure 1.

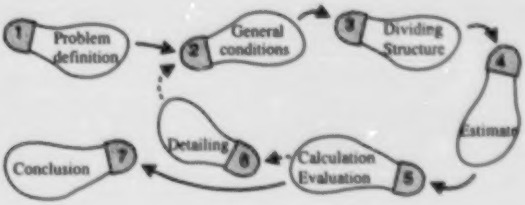

Figure 1. Elements of the iteration process in the "step-by-step" principle

3.1.1 Problem/task definition

Before the planning process can start, the project and the approach to the problem must be defined. This is important to ensure an efficient project work without disturbing discussions about irrelevant issues. To assist and clarify the definition phase, a situation map can be drawn.

3.1.2 Situation map

A situation map should highlight the essential elements of the project, such as clarity of the objective, acceptance of the objective, organisation, market conditions, innovation, complexity, size, duration. When estimating service life, other elements will be more relevant. This could be a list of the factors affecting the service life.

3.1.3 General conditions

This is a main point in the "step-by-step" principle. All external and internal factors likely to influence the project itself, as well as the project execution, must be highlighted. This should normally be done in two separate steps:

1. Brainstorm; identify all conditions of relevance
2. Select and evaluate/consider particular important conditions

In the proposed ISO factor method the factors are supposed to cover all conditions affecting the service life, including "general conditions" and synergy effects. "General conditions" and synergy effects can be numerous and difficult to account for in a reasonable and correct way. The total effect of the assumed contributions to each factor might also be rather arbitrary and hard to foresee. In section 4.2.1 we suggest a new, separate factor, in order to cover general conditions and synergy effects in a clearer and more separate manner.

3.1.4 Division, structure

The project must be divided into separate, independent parts, suitable for the individual project. In the first loop of the "step-by-step" cycle, the division should be rather coarse. In later loops (if necessary) one or more parts/groups can be divided into sub-groups, until the division is satisfactory. (This explains why the "step-by-step" expression is used.)

The seven factors in Table 2 form the first division in a service life estimation process.

3.1.5 Triple estimate

The quality of the estimates of costs/time consumption etc. is a crucial step in the "step-by-step" principle. Three values must be estimated, and preferably in the following order:

1. The minimum estimate (the 1% percentile)
2. The maximum estimate (the 99%percentile)
3. The (most) expected estimate (not necessarily the statistical expected value, see Figure 2

By focusing on the extreme estimates first, the estimates will be conservative and more realistic. Too much focus on the expected estimate could yield too narrow and definite estimates. Wide exterior values will ensure:

- A wide range of estimated values that will contain the "real value", even if this value is quite different from the most expected estimate
- Lower risk of hidden uncertainty

Good estimates can be achieved by a "neutral" (but subjective) evaluation, free of prejudice, of all input data. A group of individual persons (the resource group) are supposed to make individual estimates. After a discussion, the three values are fixed for each factor to estimate. This group process will even out individual elements of pessimism or optimism. Expertise within the resource group can be utilised for better estimates of the most expected value.

Bourke and Davies (1996) state that for practical use of the factor method, the accuracy and range of the factors should be considered, e.g. by a panel of experts for individual materials.

3.1.6 Calculation

The typical triple estimate will form a skew probability density function as shown in Figure 2.

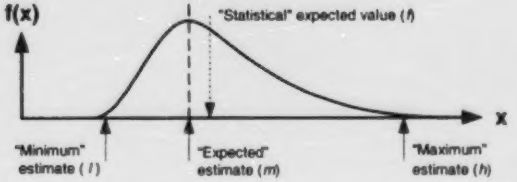

Figure 2. **Probability density function**

The "Erlang" density function with k = 10 is applied in the "step-by-step" principle, which is assumed to give a reasonably good statistical representation of the triple estimates (Klakegg 1993). The statistically expected value f and the standard deviation σ can be calculated from the following equations:

$$f = \frac{l + 2.95 \cdot m + h}{4.95} \qquad \sigma = \frac{h - l}{4.6} \qquad (1, 2)$$

where

l = minimum estimate (low)
m = expected estimate (medium)
h = maximum estimate (high)

When the plan/project has been divided into suitable parts and the triple estimates have been done for each part, the whole plan must be thoroughly calculated. Special software for this purpose has been developed at NTNU (Klakegg 1993).

When the total project values are calculated, the sum probability density function is assumed to be a normal distribution. This assumption can be done when adding up a number of independent elements of reasonably uniform size (according to the Central Limit Theorem). This assumption will not always be fulfilled, and the sensitivity of the calculations might need to be evaluated in some cases.

3.1.7 Evaluation

After the calculation, the results must be evaluated. Some questions to answer are listed below:

- Does the plan contain unreasonable or unrealistic elements (definition, division, general conditions or synergy effects)?
- Does the plan include all general conditions and synergy effects (-of relevance)?
- Is all information taken into account, that is known at the moment when the calculations are done?
- Does the result satisfy the needs?

3.1.8 Detailing and iteration

Normally it will be necessary to do one or more iterations if the plan is too coarse. Some of the elements might need to be divided into smaller sub-elements that are more clearly defined. This can enable better triple estimates.

3.1.9 Conclusion and complementary work

If the preceding analysis is carried through in a systematic and well-documented manner, much important information for future activities will be gathered. That is information for maintenance planning, quality management of design and work execution.

4 How can the "step-by-step" principle improve the proposed ISO factor method?

4.1 Possibilities

Two major aspects of the "step-by-step" principle could be useful for the service life estimation process:

1. The factors will be handled as stochastic variables. The uncertainty of each single factor will be estimated.
2. All conditions that are assumed to influence the service life can be identified and quantified in a more systematic way.

4.2 Changes to the ISO factor method

4.2.1 Removing interactions between the factors

The statistical treatment in the "step-by-step" principle is based on a structure where each cost element or factor can be assumed independent. In the ISO factor method there normally will be strong interactions between the factors. The way these effects are taken into account in the factor method gives results that are difficult to foresee. The problem can be handled in two different ways:

1. Altering the statistical treatment. This can hardly be done simply enough for practical use.
2. "Remove" every assumed synergy effect or interaction between factors. Evaluate the effect of the interactions in a new factor, such as the general factor in the "step-by-step" principle.

The latter alternative will be used in this paper, and the new factor, G, is named "general conditions".

4.2.2 Turning factors into elements and percentages into years – the element method

Several major problems entail the use of factors:

1. It is difficult to find or calculate a reasonable accurate number for the modifying factors.
2. It is difficult to evaluate the consequences of each factor in service life/years. Conversion calculations must be done in order to evaluate these.

3. The effect of multiplying factors is difficult to foresee. In the "step-by-step" principle, the estimates are added up; this is far more observable.

Problem number 1 in the section above will not be addressed further in this paper. Problem 2 and 3 may be reduced by:

1. Expressing the *estimates in years*, and
2. *Adding* up the estimates for each factor instead of multiplying them.

If the estimates are expressed directly in years it will be easier to see the effect of the estimates. The estimates should be made relative to the RSLC, namely the number of years that the given conditions are assumed to shorten and/or prolong the service life. Finally the ***l***-, ***m***-, and ***h***-estimates may be added up in order to find the ESLC and the total uncertainty, see Table 3. After such a change, the factors should be named elements. In order to avoid mixing up the two methods, the altered method will be named the element method.

5 Examples of ESLC calculations

In the ISO/CD 15686 – 1 an example is shown where the service life of a softwood window is estimated. The important conditions that influence each modifying factor are listed in a matrix. Their assumed contribution to the service life is divided into three groups: poor, average and good. 0.8, 1.0, and 1.2, respectively, is proposed as default values for the modifying factors. In the example the RSLC is 25 years, and the seven modifying factors are 1.2, 1.2, 1.2, 1, 1.2, 1 and 1.2.

5.1 ESLC calculation according to the ISO factor method

ESLC = 25 x 1.2 x 1.2 x 1.2 x 1 x 1.2 x 1 x 1.2 = 62.2 years. In the ISO-example the result is rounded off to 60 years.

5.1.1 Discussion

The factor method lacks information about uncertainty. An error range should be given for both the result (ESLC) and for the individual factors upon which the results are based.

5.2 ESLC calculation according to the "step-by-step" principle; the element method

The factors from the ISO-example will be transformed into years relative to the RSLC of 25 years. The ***m***-estimates correspond to the factors in the example above, and are found by multiplying the factors (ranging from 1.0 to 1.2) to the RSLC and then subtracting the RSLC ((25 x 1.2) – 25 = 5, and (25 x 1.0) – 25 = 0).

The ***l***-estimate is found by subtracting (0.2 x 25 = 5 years) from the ***m***-estimate. This corresponds to one step "down" in the three-divided range for factors in the matrix in the ISO example. The ***h***-estimate is found by adding 5 years to the ***m***-estimate. A more moderate addition of e.g. 2.5 years would be more realistic, since it is harder to improve a top score than reduce a high/medium score. However, by choosing a symmetric range of ± 5 years, the examples will be more comparable. For the same reason, the G-element is left inactive (set to 0).

5.2.1 Result

Table 3. Calculation sheet

			Estimates		***Statistical***			
	Elements	***l***	***m***	***h***	***calculation***	***f***	***s***	***Part of s***
A	Material and components	**0**	**5**	**10**	*f =*	5.00	2.17	14 %
B	Design	**0**	**5**	**10**	*(l+2.95 x m+h)*	5.00	2.17	14 %
C	Sitework/execution	**0**	**5**	**10**	*/4.95*	5.00	2.17	14 %
D_1	Indoor environment	**-5**	**0**	**5**	*s =*	0.00	2.17	14 %
D_2	Outdoor environment	**0**	**5**	**10**	*(h - l)/4.6*	5.00	2.17	14 %
E	Operating characteristics	**-5**	**0**	**5**		0.00	2.17	14 %
F	Maintenance level	**0**	**5**	**10**		5.00	2.17	14 %
G	General conditions	**0.0**	**0.0**	**0**		0.00	0.00	0 %
	Sum l, m, h =	*-10*	*25*	*60*	Sum ***f*** =	***25.0***		***100 %***
	RSLC + sum l, m, h =	*15*	*50*	*85*	Sqrt-sum **σ** =		***5.8***	
	ESLC, "statistical values" =	***44***	*50*	*56*	*RSLC + sum f =*	***50.00***	***ESLC =***	***50 years***
	RSLC =		**25 years**		***Result*** ⇒		***+/-***	***6 years***

Table 3 shows the estimates that are made and how the ESLC are calculated. The ESLC are 50 years ± 6 years (± 1 standard deviation or ± 12%).

5.2.2 Discussion

The element method gives a more conservative result than the factor method, due to the different nature of addition and multiplication (62 and 50 years, respectively). With a lower and more realistic ***h***-estimate, this difference would have been even bigger (an ***h***-estimate of m + 2,5 years would give a RSLC of 46 ± 4 years).

The scattering of ± 12 % indicates that the estimates are too definite and narrow. If the estimates had been made in a practical case, preferably in a resource group, the range of the estimates would most probably have been wider due to the present lack of reasonably accurate "service life" data.

As mentioned earlier, the result from the first calculation step would normally have an unacceptable amount of scattering. In this example "environment" is already divided into "indoor" and "outdoor". Another sub-division could be within the A-element, e.g. A_1 "material" and A_2 "protection" (painting). "Outdoor environment" could also be divided into "driven rain", "time of wetness", "air temperature", "pollution" etc.

Making estimates in "years" instead of in percentage of the RSLC might be confusing in some situations, e.g. when estimates are made for similar components with different RSLC. This possible drawback should be considered in further work.

6 Summary

By use of the "step-by-step" principle, as shown in the element method, the uncertainty is identified and estimated for each factor/element, as well as for the ESLC. This gives an opportunity to identify the most uncertain elements for further detailing, and for making better estimates.

The estimating process can be improved by changing the factors into elements that are added together. The estimates can also be made directly in years. These changes will make it easier to see the consequences of the estimates during the estimation process.

The statistical calculation makes the element method more complicated than the factor method. However, a computer with any kind of spreadsheet software should be available, at least for reporting, regardless of which estimating method that is used.

7 Conclusions

The "step-by-step" principle enables a stochastic handling of the modifying factors in the ISO factor method by performing a triple estimate for each factor. After the statistical calculation the estimated service life is expressed as three figures: the expected value pluss/minus one standard deviation.

The "step-by-step" principle also demonstrates a systematic approach to the estimating process, including the process of defining, subdividing and estimating the modifying factors in the ISO factor method.

8 REFERENCES

Bourke, K. and Davies H. (1996) *Factors affecting service life predictions of buildings: a discussion paper.* Watford, Construction Research Communications Ltd., England.

ISO/CD 15686 – 1 (1997) *Buildings: Service Life Planning, part 1 – general principles.*

Klakegg, O.J. (1993) *Trinnvisprosessen.* Dept. of Building and Construction Engineering, NTNU, Norway (in Norwegian).

Klakegg, O.J. (1994) *The step-by-step principle – a systematic approach to project planning under uncertainty".* Department of Building and Construction Engineering, NTNU, Norway.

Lichtenberg, S. (1990) *Projektplanlægning i en foranderlig verden.* 2. Ed. Polyteknisk forlag, Lyngby, Denmark, (in Danish).

NS 3420 (1972) *Beskrivelsestekster for bygg og anlegg.* Norwegian standard, Norges Byggstandardiseringsraad, NBR, Oslo, Norway, (in Norwegian)

Siemes, T. (1997) *Probability and reliability in service life prediction.* TNO Building and Construction Research, Delft, The Netherlands.

117 DEVELOPMENT OF PREDICTION MODELS FOR SEWER DETERIORATION

D.M. Abraham and R. Wirahadikusumah

Purdue University, W. Lafayette, IN, USA

Abstract

Due to their low visibility, rehabilitation of sanitary sewers is often neglected until catastrophic failures occur. Neglecting regular maintenance of these underground utilities adds to life-cycle costs and liabilities, and in extreme cases, stoppage or reduction of vital services. Incorporating condition data and deterioration patterns of the city's sewer system is pivotal for obtaining a realistic assessment of the city's infrastructure. This paper will explore the probability-based Markovian approach for modeling deterioration. This approach is based on the assumption that since the behavior of sewer lines (i.e., the rate of deterioration) is probabilistic, the selection of an appropriate repair strategy is also an uncertain procedure. Probability-based prediction models enable the comparison of the expected proportions in given condition states with the actual proportions observed in the field, and in this way possible defects in construction, materials, quality control, etc., can be identified. Expert opinions from engineers, who have developed the sewer assessment surveys for the City of Indianapolis Department of Capital Asset Management (DCAM), will be used to validate the deterioration models developed in the research. More realistic deterioration models will assist asset managers in improved performance modeling of the sewer infrastructure and also in determining this infrastructure's rehabilitation costs based on improved estimates of deterioration.

Keywords: deterioration, sewer systems, probabilistic methods, condition states, Markovian models, infrastructure, rehabilitation, maintenance

1 Introduction

The need to renew the deteriorating civil infrastructure in the U.S. creates a tremendous challenge to civil infrastructure system planners and managers in terms of financing, logistics, and demand and service assessment. Underground infrastructure such as sewer lines have to compete with other types of facilities for limited funds. Very often, maintenance of these facilities are neglected. Many of the nation's sewer systems are deteriorating and vulnerable to failure. Some failures can endanger public

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health and safety. Reactive maintenance strategies often result in difficult and costly rehabilitation (WEF-ASCE 1994). Tremendous capital can be protected if the cities could make better investment decisions and consider infrastructure management on a proactive basis.

Incorporating condition data and deterioration patterns of the city's sewer system is pivotal for obtaining a realistic assessment of the city's infrastructure. Deterioration models will assist asset managers in improved performance modeling of the sewer infrastructure and in determining this infrastructure's rehabilitation costs based on more realistic estimates of deterioration.

This paper will discuss the mechanisms of structural failures in sewers. An overview of prior research in the area of deterioration modeling for other infrastruture systems (e.g., bridges, pavements) will be provided. Finally, the use of Markov chain models for deterioration modeling of sewer systems will be described.

2 Deterioration in Sewer Systems

Sewer collapses are caused by hydraulic failures and structural failures. Hydraulic failures are caused by infiltration and inflow (I/I) problems. Infiltration is water that enters the system from the ground through pipe defects, while inflow is extraneous storm water that enters the system through roof leaders, direct stormwater connections, clean-outs, foundation drains, basement sump pumps, etc. These I/I problems reduce the planned hydraulic capacity of sewers, increasing the potential for collapse.

The intensity of structural failures depend on the size of defects, soil type, interior hydraulic regime, groundwater level and fluctuation, method of construction, and loading on sewers.

2.1 Materials

The stability of deteriorated sewers depends on the materials used for the construction of the sewer pipes. Rigid pipe materials are usually designed to resist vertical loading on their own, while brick sewers and flexible pipe materials require side support from the surrounding soil. Older sewers were typically constructed of vitrified clay, brick, or concrete. Presently, materials such as plastic, ductile iron, steel, reinforced concrete, reinforced fiberglass, etc., are used.

Extensive research projects which investigated all aspects of concrete and brick sewer system performance have been performed by Water Research Center (WRC) in order to understand the modes of failure for sewers (Serpente 1993). The research included field, laboratory, and theoretical studies. Two-hundred-and-fifty cases of sewer collapse were examined and mechanisms of ground loss and void formation were investigated. Based on these studies, it was found that the mortar and materials in pipe joints can be eroded, and corrosion in concrete sewers can occur due to the presence of hydrogen sulfide or other chemicals.

2.2 Soil conditions

The state of the surrounding soil is of great importance in assessing the structural condition of a sewer. The main factors that affect the rate of ground loss include sewer defect size, hydraulic conditions (water table, frequency and magnitude of surcharge), and soil properties (cohesive or non-cohesive soil). Severe defects (larger than 10 mm), high water table (above sewer level), frequent and high magnitude of hydraulic surcharge, and soil types: silts, silty fine sands, fine sands, can have serious effects on ground loss. Loss of side support will cause the side of the pipe to move outward when loaded vertically and collapse will be likely once the pipe deformation exceeds 10%. Uneven loading of pipes due to joint displacement also accelerates the pipe deterioration process.

3 Prior Research in Deterioration Modeling and Defects Analysis for Infrastructure Systems

Pavements and bridge assessment and deterioration models have been the basis for research in infrastructure systems.

3.1 Pavement Systems

Pavement deterioration models, currently in use, vary in complexity from simple straight-line extrapolation to probability models (Butt, et. al. 1994). However, when sufficient data are available, it is found that the shape of the deterioration curve is generally curvilinear rather than straight-line. Straight-line extrapolation tends to produce unrealistic results, while regression techniques are valid only if predictive variables that are related to condition deterioration can be found.

Probability-based Markov models have been used to portray the non-deterministic behavior of pavements. This form of predictive model has the advantage of ensuring that projections beyond the limits of data will continue to have the worsening pattern with age, something that regression models cannot guarantee (Butt, et. al. 1994). Other research studies on pavement deterioration models includes the utilization of latent variables to model characteristics that are not easily measurable or directly observable. Ben-Akiva and Gopinath (1995) propose a methodology that estimates deterioration using a latent dependent variable whose value is estimated from a set of indicators, which are measures of damage on the pavement. This modeling approach claims to have the flexibility to include different measurement techniques and data collection strategies.

3.2 Bridge Systems

Deterioration models developed for bridge systems include linear, piece-wise linear, linear regression coupled with a spike reflecting a single rehabilitation, non-linear regression with exponential decay functions coupled with spikes to reflect the effects of rehabilitation, and methods that incorporate Markov chain. Linear models do not provide sufficient accuracy, while the effort to develop the Markov chain requires an extensive database. Sanders and Zhang (1994) present deterioration models and modifications to databases that produce reasonable projections for bridge management systems that have limited data. Lu and Madanat (1994) have developed a method that employs condition data collected during facility inspections to improve the precision of deterioration models. The updating method used is a Bayesian approach, which combines prior information and sample information. This method decreases the level of uncertainty in forecasting the condition of bridge structures, and permits the development of a deterioration model even with limited data.

3.3 Sewer Systems

Several recent studies, including EPA (1991a), EPA (1991b) and ASCE (1989), have explored critical sewer management issues in order to provide information to support sanitary sewer infrastructure design and management. Dillard, Kurz and Stonecipher (1993) discuss the development of a defects classification system for sewers in order to provide objective criteria for evaluating most commonly observed defects and selecting proper rehabilitation measures. This classification system, used by the Metropolitan Department of Water and Sewerage Services (MWS) in Nashville, Tennessee, claims to improve correction of identified problems while minimizing their potential transfer to adjoining line segments.

Fick, Johnson and Johnson (1993) stress that effective management of urban infrastructure requires accurate assessment of system-wide condition. Their paper describes the development of a computer model to provide information about the overall condition of the sanitary sewer system and the associated costs of rehabilitation. The system developed by CH2M Hill in cooperation with the City of San Jose, California, provides an estimate of the system-wide condition through the inspection of only a fraction of the pipeline population. Sewer pipes are rated according to corrosion and structural conditions observed during closed-circuit TV (CCTV) inspections. However, the assessment is based primarily on the observed condition of the sewer system, and deterioration patterns are not considered.

Other work in the area of sanitary system assessment and management include the investigation of methods for collection of multi-sensory data for sewer inspection systems (Gokhale et. al. 1997, Abraham, et. al. 1997), and methods for automating the interpretation of the multisensory data (Wirahadikusumah, et. al. 1997). The understanding of sewer deterioration mechanisms helps asset managers in developing a prediction model by estimating whether a sewer has deteriorated sufficiently for collapse to be likely. One approach that has been successfully used in other types of infrastructure deterioration modeling is the Markov chain model. The following sections describe the initial results of the development and validation of a Markov chain model for predicting sewer deterioration.

4 Use of Markov Chain Model

Stochastic processes are processes that evolve over time in a probabilistic manner. In the sewer deterioration model, the stochastic processes $X_1, X_2, X_3, \ldots$ can represent the collection of condition ratings of a sewer line based on each five-year inspection data. At any time t, the condition of a sewer line can be described in exactly one of a finite number of mutually exclusive and exhaustive categories or *states*. In this case, five states (associated with condition ratings 1 to 5, with 1 being optimal sewer condition and 5 corresponding to critical sewer condition) are used.

A stochastic process is a Markov chain if it has the Markovian property, i.e., the conditional probability of any future event, given any past event and the present state $X_t = i$, is independent of the past event and depends only upon the present state. This property can be expressed as follows:

$$P(X_{t+1} = i_{t+1} \mid X_t = i_t, X_{t-1} = i_{t-1}, \ldots, X_1 = i_1, X_0 = i_0) = P(X_{t+1} = i_{t+1} \mid X_t = i_t) \qquad \ldots (1)$$

In order to reduce the complexity of the analysis, the future condition of sewers is assumed to depend only on the present state, and independent of the past condition. It is further assumed that for all states i and j and all t, $P(X_{t+1} = i_{t+1} \mid X_t = i_t)$ is independent of t. The probability, p_{ij}, that sewer condition is in state i at time t and it will be in a state j at time t+1 does not change (i.e., it remains stationary) over time (unless rehabilitation is performed, or other external factors change). This *stationary* assumption is expressed by equation (2).

$$P(X_{t+1} = j \mid X_t = i) = p_{ij} \quad \text{... (2)}$$

The term *transition* is used when the system moves from state i during one period to state j during the next period. Accordingly, the probabilities, p_{ij}'s, are referred to as the *transition probabilities*. The transition probabilities are commonly displayed as an m x m matrix called the transition probability matrix P. In this study, there are five states associated with the five possible conditions of sewers (state 1 corresponds to the optimal condition and state 5 corresponds to the worst condition). To simplify the computation, it is assumed that sewer deteriorate by one state in one transition period. Thus, the transition probability matrix is given by:

$$P = \begin{bmatrix} p_{11} & p_{12} & 0 & 0 & 0 \\ 0 & p_{22} & p_{23} & 0 & 0 \\ 0 & 0 & p_{33} & p_{34} & 0 \\ 0 & 0 & 0 & p_{44} & p_{45} \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix} \quad \text{... (3)}$$

The aforementioned transition probability matrix is for one-step (one-period) transition. The *n-step transition probability matrix*, $P^{(n)}$, of the process that is in state i and will be in state j after n periods (as opposed to in the *next* period) is computed using Chapman-Kolmogorov equation: $P^{(n)} = P^n$. The n-step transition probability matrix is then obtained by taking the n-th power of the one-step transition matrix (Ross 1997).

This Markovian model provides a reliable mechanism for developing prediction models. The Markov process imposes a rational structure on the deterioration model because it explains the deterioration as uncertain, and it also ensures that the projections beyond the limits of data will continue to have worsening condition pattern with age.

5 Example of Application

A significant requirement for the successful application of Markov chain model is the availability of reasonably good estimates of probability values in the transition matrices. These probability values can be estimated by analyzing deterioration curve for each sewer classification. These curves can be developed based on the historical records of sewer conditions, or based on expert opinions and assumptions.

In this study, expert opinions from the engineers who were instrumental in developing the sewer assessment surveys for the City of Indianapolis, Indiana, were used to estimate the probability values for the transition matrices. The study focused on large 60-inch (1500 mm) or larger combined sewers. For modeling purposes, the sewers were classified in terms of the following:

- Material: flexible sewers (brick, clay), rigid sewers (concrete, reinforced concrete)
- Ground water table (GWT): high ground water table (pipe crown is below GWT), low ground water table (pipe crown is above GWT)
- Soil type: cohesive soil, non-cohesive (granular) soil

- Depth cover: normal depth (between 3 feet (900 mm) and 20 feet (6000 mm), too shallow and under high live load (less than 3 feet (900 mm)) or too deep (more than 20 feet (6000 mm)).

Development of models for predicting deterioration in flexible sewers is discussed. The ideal conditions for flexible sewers are low ground water table, non-cohesive soil for backfill, and normal depth. Under these ideal conditions, the expected life of flexible sewers is about 100 years. The deterioration curve based on expert opinions for flexible sewers in ideal conditions is shown in Figure 1.

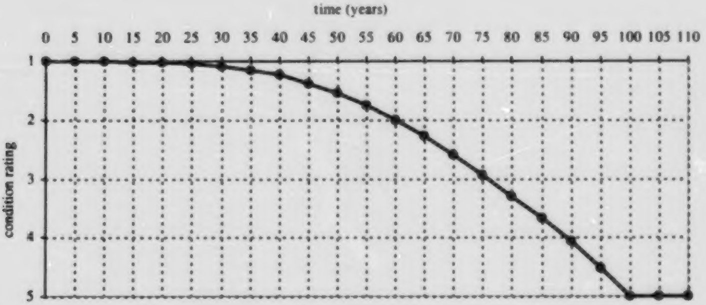


Figure 1: Deterioration curve of flexible sewers under ideal conditions

While flexible sewers in ideal conditions will reach their worst condition (rating 5) in 100 years, the expected life of similar sewers in adverse conditions such as very shallow depth cover, is decreased to about 90 years. However, the experts believe that the shape of the deterioration curve is unchanged. Table 1 shows the expected life of flexible sewers under different conditions, based on expert opinions.

Table 1: Expected life of flexible sewers

Soil type of backfill	Ground water table	Depth of cover	Expected life (years)
Non-cohesive	Low	Normal	100
Non-cohesive	Low	Too shallow or too deep	90
Non-cohesive	High	Normal	80
Non-cohesive	High	Too shallow or too deep	70
Cohesive	Low	Normal	65
Cohesive	Low	Too shallow or too deep	60
Cohesive	High	Normal	45
Cohesive	High	Too shallow or too deep	40

To estimate the probability values in transition matrices, a non-linear optimization technique is used to minimize the sum of absolute difference between the data points (from Figure 1) and the predicted condition for the corresponding age generated by the Markov chain model. The objective function of the non-linear optimization has the following form:

$$\text{Minimize} = \sum_{t=1}^{N} | Y(t) - E[X(t,P)] | \qquad \ldots (4)$$

where N = total number of periods
Y(t,j) = sewer condition data point at time t (from Figure 1)
E[X(t,P)] = expected value of sewer condition at time t as predicted by the Markov chain model with probability matrix P.

In this problem, the decision variable is the transition matrix P. As previously described, P is a 5x5 matrix as shown in (3). The expected condition of sewer at time t=n (which is in state 1 at time t=0), E[X(t=n,P)], is calculated using (5).

$$E[X(t=n,P)] = [1\ 0\ 0\ 0\ 0]\ P^{(n)}\ [1\ 2\ 3\ 4\ 5]^T \qquad \ldots (5)$$

To account for the assumption that older sewers would deteriorate faster, different transition matrices are used for every five-stage zone. By dividing the life of sewers into five-stage zones, there is an added advantage that the computation of the complex objective function shown in (4) becomes reasonably manageable.

Suppose the transition probability matrix for the first five stages (stage 0 to 5) is P and for the next five stages (stage 6 to 10) is Q. Then the expected condition of sewer at time t = 6 is estimated by

$$E[X(t=6,P)] = [1\ 0\ 0\ 0\ 0]\ P^{(5)}\ Q^{(1)}\ [1\ 2\ 3\ 4\ 5]^T \qquad \ldots (6)$$

As an example, consider flexible sewers in cohesive backfill, high ground water table, and deep cover. The transition matrices P and Q for such sewers that minimize equation (4) are shown in (7) and (8). Figure 2 shows the deterioration curve, as predicted by Markov chain model, for flexible sewers in such conditions. The expected life of such sewers is forty years.

$$P = \begin{bmatrix} 0.8500 & 0.1500 & 0 & 0 & 0 \\ 0 & 0.4696 & 0.5304 & 0 & 0 \\ 0 & 0 & 0.0001 & 0.9999 & 0 \\ 0 & 0 & 0 & 0.0001 & 0.9999 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix} \qquad \ldots (7)$$

$$Q = \begin{bmatrix} 0.0001 & 0.9999 & 0 & 0 & 0 \\ 0 & 0.0001 & 0.9999 & 0 & 0 \\ 0 & 0 & 0.0001 & 0.9999 & 0 \\ 0 & 0 & 0 & 0.2462 & 0.7538 \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix} \qquad \ldots (8)$$

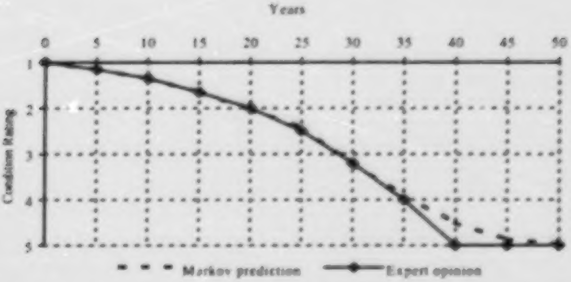


Fig. 2: Deterioration curve of flexible sewers in cohesive backfill, high ground water table, and deep cover.

The probability values in the transition matrices for flexible sewers in various conditions have been estimated using the same techniques. Table 2 lists the results and Figure 3 shows the comparison between deterioration curves based on expert opinions (3.a) and those developed by Markov chain models, computed with non-linear programming techniques (3.b).

Table 2: Estimated transition probabilities

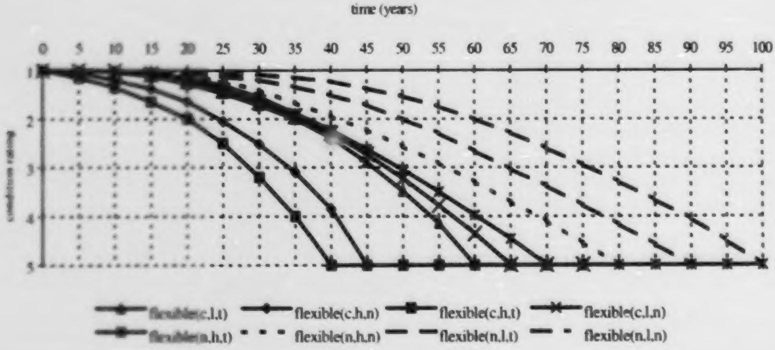
	cohesive, high, too deep expected life: 40 years				cohesive, high, normal depth expected life: 45 years			
	p_{11}	p_{22}	p_{33}	p_{44}	p_{11}	p_{22}	p_{33}	p_{44}
stage 1-5	0.8500	0.4696	0.0001	0.0001	0.9226	0.2567	0.0001	0.0001
stage 6-10	0.0001	0.0001	0.0001	0.2462	0.3671	0.1536	0.0001	0.0001

	cohesive, low, too deep expected life: 60 years				cohesive, low, normal depth expected life: 65 years			
	p_{11}	p_{22}	p_{33}	p_{44}	p_{11}	p_{22}	p_{33}	p_{44}
stage 1-5	0.9676	0.2917	0.0001	0.0001	0.9759	0.2929	0.0001	0.0001
stage 6-10	0.7523	0.4720	0.0001	0.0001	0.6943	0.6629	0.0972	0.0001
stage 11-15	0.0001	0.0001	0.0001	0.0001	0.0001	0.0845	0.0001	0.0001

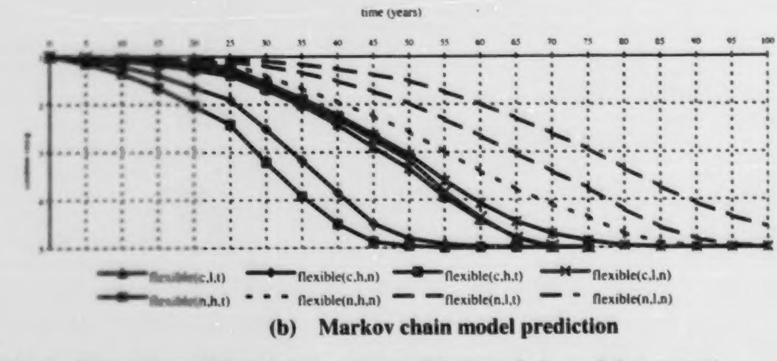
	non-cohesive, high, too deep expected life: 70 years				non-cohesive, high, normal depth expected life: 80 years			
	p_{11}	p_{22}	p_{33}	p_{44}	p_{11}	p_{22}	p_{33}	p_{44}
stage 1-5	0.9817	0.3011	0.0001	0.0001	0.9886	0.3073	0.0001	0.0001
stage 6-10	0.6845	0.6339	0.4410	0.0688	0.7347	0.7546	0.3819	0.4432
stage 11-15	0.2839	0.3125	0.3002	0.1962	0.5934	0.5548	0.4139	0.4124
stage16-20	N/A	N/A	N/A	N/A	0.0001	0.0001	0.0001	0.0001

	non-cohesive, low, too deep expected life: 90 years				non-cohesive, low, normal depth expected life: 100 years			
	p_{11}	p_{22}	p_{33}	p_{44}	p_{11}	p_{22}	p_{33}	p_{44}
stage 1-5	0.9920	0.3122	0.0001	0.0001	0.9957	0.5309	0.0001	0.0001
stage 6-10	0.9145	0.4786	0.1765	0.0001	0.9613	0.4460	0.0001	0.0001
stage 11-15	0.7383	0.4363	0.3193	0.0001	0.8138	0.4510	0.5250	0.0001
stage16-20	0.1947	0.0001	0.0001	0.0001	0.5345	0.3153	0.4130	0.3535

Deterioration curves developed by Markov chain model (Figure 3.b.) give good approximations to those obtained from expert opinions (Figure 3.a.). The application of Markov chain model in developing deterioration curves also ensures that the projections beyond the limits of data will continue to have worsening condition pattern with time.



(a) Based on expert opinions



(b) Markov chain model prediction

Fig. 3: Comparison of deterioration curves (Markov chain model vs. expert opinions)

6 Summary

Effective deterioration modeling will assist asset managers in improved performance modeling and more accurate prediction of the condition of sewer infrastructure, and will be a step towards preventive rather than reactive maintenance. This paper discussed the use of Markov chain models for developing deterioration models for flexible sewer systems under different soil and groundwater conditions. The deterioration curves developed closely resembled the curves obtained based on expert opinions. Future work will focus on integrating the deterioration models, with automated sewer condition assessment models, as well as with infrastructure management systems.

7 Acknowledgements

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8 References

Abraham, D. M., Iseley, T., Prasanth, R. K. and Wirahadikusumah, R. (1997). Integrating Sensing Technologies for Underground Utility Assessment. *ASCE Conference on Infrastructure Condition Assessment: Art, Science, Practice*, Boston, Massachusetts, August 1997, 316-325.

ASCE (1989). Sulfide in Wastewater Collection and Treatment Systems, *ASCE Manuals and Reports on Engineering Practice No. 69*, Joint Committee of the American Society of Civil Engineers and the Water Environment Federation, New York.

Ben-Akiva, M., Gopinath, D.(1995) Modeling Infrastructure Performance and User Costs. *ASCE Journal of Infrastructure Systems*, 1(1), 33-43.

Butt, A.A., Shahin, M.Y., Carpenter, S.H., and Carnahan, J.V. (1994) Application of Markov Process to Pavement Management Systems at Network Level. *Proceedings of The Third International Conference on Managing Pavements*, TRB-NRC, 159-172.

Dillard, C.W., Kurz, G.E., and Stonecipher, P.A. (1993) Management of Sewer System Rehabilitation for the Overflow Abatement Program in Nashville, Tennessee. *Proceedings of the International Conference on Pipeline Division of the ASCE*, San Antonio, Texas, 449-467.

EPA (1991a) *Handbook: Sewer System Infrastructure Analysis and Rehabilitation*, United States Environmental Protection Agency, 625/6-91/030.

EPA (1991b) Technology Transfer Seminars: Sewer System Infrastructure Analysis and Rehabilitation, United States Environmental Protection Agency, Cincinnati, Ohio.

Fick, B., Johnson, M., and Johnson J. (1993) Predictive Model Aids in Development of Systemwide Rehabilitation Program. *Proceedings of the International Conference of Pipeline Division of the ASCE*, San Antonio, Texas, 616-627.

Gokhale, S., Abraham, D. M., Iseley, T. (1997) Intelligent Sewer Condition Evaluation Technologies - An Analysis of Three Promising Options. *North American No-Dig '97 Conference*, Seattle, April 1997, 253-265.

Lu, Y. and Madanat, S. (1994) Bayesian Updating of Infrastructure Deterioration Models. *Transportation Research Record 1442*, Transportation Research Board - National Research Council, 110-114.

Ross, Sheldon M. (1997) *Introduction to Probability Models*, 6th edition, Academic Press.

Sanders, D. H. and Zhang, Y. J. (1994) Bridge Deterioration Models for States with Small Bridge Inventories. *Transportation Research Record*, 1442, 101-109.

Serpente, R. F. (1993) Understanding the Models of Failure for Sewers. *Proceedings of the International Conference on Pipeline Division of the ASCE*, San Antonio, Texas, August 1993, 86-100.

WEF-ASCE (1994) *Existing Sewer Evaluation and Rehabilitation*, ASCE Manuals and Reports on Engineering Practice, No. 62, Alexandria, Virginia.

Wirahadikusumah, R., Abraham, D. M., Iseley, T., Prasanth, R. K. (1997) Assessment Technologies for Sewer System Rehabilitation. *Journal of Automation in Construction*, 7(1998), 259-270.

118 SERVICE LIFE WITH REGARD TO FROST ATTACK — A PROBABILISTIC APPROACH

G. Fagerlund

Lund Institute of Technology, Lund, Sweden

Abstract

Significant frost damage will not occur until a certain critical moisture level is transgressed over a sufficiently big portion of the structure. The critical moisture level is a "fracture value" which can be compared with the load carrying capacity in structural design. It is a materials property that seems to be rather uninfluenced by normal variations in environmental properties, such as number of freeze-thaw cycles and minimum freezing temperature. The moisture content inside the structure depends on the outer moisture conditions; the more moist the environment, the larger the inner moisture content, and the larger the risk of frost damage. The actual moisture content in the structure can be compared with the actual load in structural design. The risk of frost damage can be calculated when the frequency functions of the two parameters, critical moisture content and actual moisture content are known. Some hypothetical cases are treated in the paper showing that the probability of frost damage might actually decrease with increasing exposure time in moderately moist environments, but that it normally increases with increasing exposure time in continuously moist environments where the structure has no possibility to dry.

Keywords: Frost resistance, service life

1 Frost attack generally – "The critical moisture content"

Consider a small but representative material volume inside a structure, Fig. 1(a). For most building materials the size of a representative volume (unit volume) is less than 1 cm^3. The material is supposed to be porous with the porosity P, (m^3/m^3). The structure is exposed to moisture from outside so that a certain moisture profile occurs over the cross-section of the structure, Fig. 1(a). The size

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of the representative (unit) volume is so small that its moisture content we, (kg/m^3), can be assumed to be constant. Thus a degree of saturation

$$S= w_e/(1000\ P) \qquad \textbf{(1)}$$

S, (m^3/m^3) of the unit volume can be defined
where 1000 kg/m^3 is the density of pore water.

All neighbouring unit volumes have almost the same degree of saturation under such natural situations where the outer environment is very moist and, therefore, the risk of frost damage is imminent. As a first approximation, this means that each unit volume can be treated as an isolated material volume which is not affected by its "neighbours" when frost action occurs. No more water is transferred to the unit volume during freeze/thaw than transferred from the volume.

If the material is exposed to frost, some pore water is transformed into ice. This causes stresses inside the unit volume considered. Theoretically, all principal damage mechanisms proposed (e.g. Powers 1949, Powers and Helmuth 1953) will lead to the occurrence of a critical, or maximum, allowable moisture content inside the unit volume, (Fagerlund 1979, 1996). In the following this moisture content is expressed in terms of a critical degree of saturation, Scr. The theoretical prediction of the existence of a critical moisture content has been verified by numerous experiments, (e.g. Fagerlund, 1972, Klamrowski and Neustupny, 1984).

Theoretically, and also experimentally, it can be shown that the value of Scr is hardly influenced by changes in the freezing rate (Fagerlund 1992, Klamrowski and Neustupny, 1984). It has also been shown experimentally that it is almost unaffected by an increased number of freeze-thaw cycles (Fagerlund, 1973, Rombén 1974, Klamrowski and Neustupny, 1984). On the other hand, since the amount of freezable water increases with decreasing temperature, the Scr-value ought to be somewhat influenced by the lowest temperature reached during each freeze-thaw cycle. It seems safe to state, however, that the value of the critical moisture content is a true materials property that is almost constant when the material is used during normal, natural variations in the outer climate.Thus, Scr can be compared with the strength, or fracture stress, used in structural design.

Examples of experimental determinations of the critical moisture content by freeze-thaw tests are shown in Fig. 2. The critical value is indicated by the rather drastic change-over from undamaged to damaged material within a narrow range of degree of saturation. It should be noted that the specimens in Fig. 2 have only been exposed to 3 freeze-thaw cycles with constant degree of saturation. Therefore, it is evident that few freezings cause considerable damage when the material is more than critically saturated.

The Scr-value can also be determined theoretically assuming the existence of a maximum distance inside the material for moisture flow during freezing, (Fagerlund 1979). Other factors besides the maximum distance determining the Scr-value are:

1. The total content of coarse pores (radius bigger than about 5 μm)
2. The size distribution of such coarse pores
3. The manner by which the coarse, initially air-filled, pore system is inactivated by water absorption when the material is more or less continuously exposed to water from an outside source

The Scr-value of the unit cell is of course not precisely defined. It has a certain variation depending on variations in the materials properties. To some extent it is also influenced by natural variations in the outer climate, such as the number of freeze-thaw cycles and the minimum freezing temperature. Therefore, Scr can be described by a frequency function f_{Scr} or a distribution function F_{Scr}. Under ideal conditions these are normal distributed.

For many materials Scr can be assumed to be independent of time, simply because the material itself does not change with time. For other materials, such as concrete, there is a gradual hydration changing the properties that are determining the value of Scr. Therefore, one can assume that there is a certain time dependency also in Scr, so that the frequency and distribution functions can be described by f_{Scr}=f[Scr(t)], and F_{Scr} = F[Scr(t)]. These principles are illustrated in Fig. 1(b).

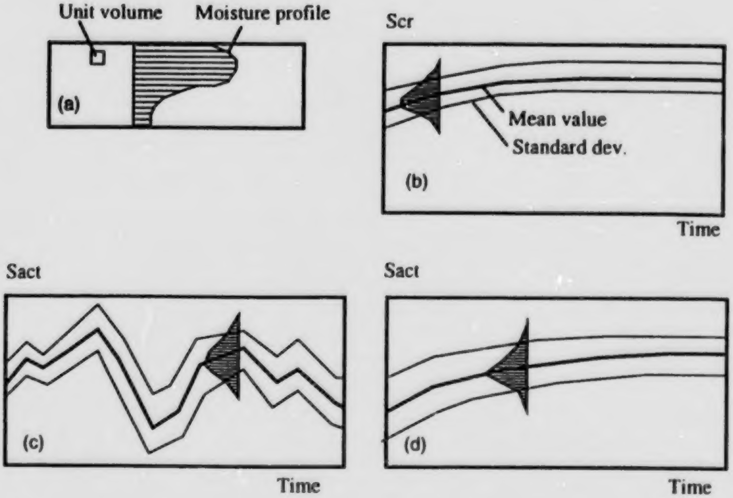


Fig. 1:

(a) A representative unit volume inside a structure and the moisture profile across the structure

(b) The critical degree of saturation of the unit volume, and its distribution, as function of time

(c) The actual degree of saturation of the unit volume, and its variation, as function of time. Varying outer moisture conditions

(d) The actual degree of saturation of the unit volume, and its variation, as function of time. Permanent exposure to moisture at the surface

The frequency function f_{Scr} can be determined experimentally by freeze-thaw tests of a representative number of specimens. The freeze-thaw cycle used can either be a standard cycle representing the most severe cycle likely to occur in practice, or a spectrum of freeze-thaw cycles representing the real spectrum occurring in practice. However, the effect of the shape of the freeze-thaw cycles on f_{Scr} will probably not be so big. An experimental technique for the determination of Scr has been developed (RILEM 1977).

An example of a determination of the time effect on the Scr-value of concrete is shown in Fig. 2. In this case the Scr-value is increasing with time.

A special type of frost attack is salt scaling ocurring when the material is exposed to combined frost and salt solution at the surface. Typical solutions are sea water and de-icing agents. Under such conditions, surface spalling often occurs. This type of frost attack is caused by moisture ingress during the freeze/thaw cycle. The theory presented in this paper does not cover this type of attack.

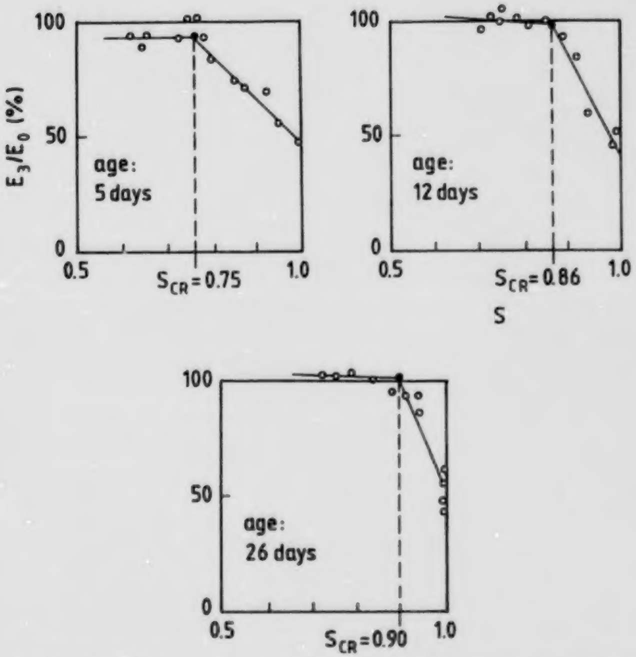


Fig. 2: The critical degree of saturation of a certain concrete as function of the concrete age. The parameter E3/Eo is the ratio dynamic E-modulus after 3 freeze-thaw cycles and before freeze-thaw

2 The water content in practice – "The actual moisture content"

In practice, the unit volume will take up moisture. Its moisture content can be expressed in terms of an actual degree of saturation, Sact where S is defined by Eqn 1. Sact will depend on the pore size distribution and on the "wetness" of the environment. One can theoretically show that pores that are smaller than about 1 μm will become completely saturated after a rather short time of exposure to free water, while it takes a much longer time to fill the coarse pore system, (Fagerlund 1996). But, for most materials, the Scr-value corresponds to a certain water-filling of the coarse pore system. Therefore, frost damage will not occur until the unit volume has had time to fill itself by water to more than the critically saturated condition. This takes time, and it means that the risk of frost damage increases with increased possibility for the material to take up water, and decreased possibility to dry. Therefore, the longer the exposure to free water present at the material surface, the bigger the risk of frost damage.

In the real case, with given environmental conditions as wetness, the Sact-value will evidently be a function of time. It is quite clear that the Sact-value will be a stochastic variable that can be expressed in terms of a frequency function, f_{Sact} and a distribution function, F_{Sact}. These principles are shown in Fig. 1(c). The variation in Sact will depend on variations in the pore structure, but above all on variations in the outer climate. It is safe to assume that the variation in Sact is considerably larger than the variation in Scr simply because the normally occurring large variations in the outer climate is fundamental for Sact but of marginal importance for Scr. For many types of environment the functions f_{Sact} and F_{Sact} are independent of time, seen over a given constant time interval, e.g. one year. Each year gives about the same spectrum of Sact. The two functions only depend on natural climatic conditions. Typical environments giving this type of behaviour are:

- A façade exposed to normal variations in rain and outer relative humidity. The outer climate generates an internal climate expressed in terms of a variation in the Sact-value of the unit volume. This variation is about the same each year.
- A road surface exposed to normal variations in rain, melting snow and RH. Each new winter will give about the same variation in the internal Sact-value as the previous winter, or the winters to come.

Other types of environment will give a steadily increasing value of Sact with increasing exposure time, Fig. 1(d). Typical examples are:

- A concrete dam wall constantly exposed to free water, which will cause a gradual increase in the Sact-value of each unit volume inside the wall.
- A concrete pier standing in water, in the splash zone.
- A ground-wall founded below the ground water level and, therefore, constantly sucking ground water.

In these cases, the functions f_{Sact} and F_{Sact} will be time dependent, $f_{Sact} = f[Sact(t)]$ and $F_{Sact} = F[Sact(t)]$. In the simpliest case, the standard deviation of Sact can be assumed to be constant while the mean value of Sact increases with increasing time.

It is a very difficult task to determine the exact shape of the function f_{Sact} since it involves a calculation or measurement of the relation between the outer climate (frequency of rain and snow, frequency of RH, sequence of rain and RH, etc) and the internal moisture content in all parts of the structure. It is not possible to have all the required information concerning the outer climate. Therefore, one has to use a standard environment representing the real environment as closely as possible. Such an environment could be a cyclic variation composed of fixed periods of free water at the surface and fixed periods of a certain constant RH at the surface. Then, by using computer simulations, or by measurements on the structure, one can estimate the moisture variation inside the structure. A technique for calculating the moisture content in concrete at such cyclic variations of the outer environment has been developed (Fagerlund and Hedenblad 1993).

Other complications in estimating the function f_{Sact} are:

- Frost damage almost always occurs when the moisture content is above the so-called hygroscopic range. It is very difficult to calculate moisture transport at these very high moisture levels. Ordinary transport equations, transport data and moisture fixation data used in the hygroscopic range cannot be used. A technique for the determination of the relevant material properties at these high moisture contents, together with a calculation method, has been developed (Janz 1997).
- There are no good methods for the determination of the moisture content above the hygroscopic range. Therefore, it is difficult to verify the calculation methods. Methods such as NMR can be used, but they are limited to laboratory test and are very expensive. Weight measurements are fairly precise, but they are destructive. One theoretical possibility is to use an osmotic cell by which it ought to be possible to measure the capillary under-pressure, and thereby indirectly the moisture content, but so far no cells have been developed. Electrical methods (resistive or capacitive) might be used, but they are very sensitive to salt migration in the material, or to other disturbances.

An alternative method, used by the author, is to expose at thin slice of the material to continuous moisture uptake from one side. Then, a constantly increasing moisture uptake will occur, which can be detected by weighing the slice at regular intervals. By exposing a number of slices representing the natural variations in materials properties one will obtain a spectrum of Sact-values which can be used for calculating the function f_{Sact}. This technique will give the maximum values of Sact likely to occur in practice. One can only continue the measurements for 1 or 2 months at maximum. Therefore, one has to make an extrapolation of the measurements to longer absorption times. A theoretical analysis indicates that the Sact value can be described by the following type of equation (Fagerlund 1996):

$$Sact = A+B\,t^{C} \qquad (2)$$

where A, B and C are coefficients determined by the short-term absorption experiment; t is the absorption time. The coefficient C is smaller than 0.5. Therefore, the water absorption is gradually slowing down with time. It is reasonble to assume that the function f_{Sact} will have the same shape irrespectively of the absorption time. The only effect of time is that the mean value of Sact increases with time according to Eqn 2.

3 The probablity of frost damage

The unit volume will be frost damaged if freezing occurs in the more than critically saturated volume. Thus, the condition for frost damage to occur can be defined by:

$$Sact>Scr \tag{3}$$

The unit volume will be severely damaged also in cases where Scr is transgressed by a very small amount. Besides, one single freezing is enough to cause damage. Severe frost damage will not occur in a structure, if only one single unit volume is damaged, or only a few unit volumes. For macroscopic frost damage to occur, a considerable volume fraction of the structure must be more than critically saturated. It seems reasonable to assume, however, that this often happens; viz. if one unit volume can be damaged it is reasonable to assume that also adjacent unit volumes can be damaged at the same time, since their moisture contents ought to be about the same. Therefore, the criterion expressed by Eqn 2 can be assumed to be valid also for the material in bulk.

Both Sact and Scr are stochastic variables. The probability P{Scr<Sact} that frost damage occurs can be calculated by:

$$P\{Scr<Sact\} = \int_0^1 F_{Scr}\, f_{Sact}\, dS \qquad 0 \le P[Scr<Sact] \le 1 \tag{4}$$

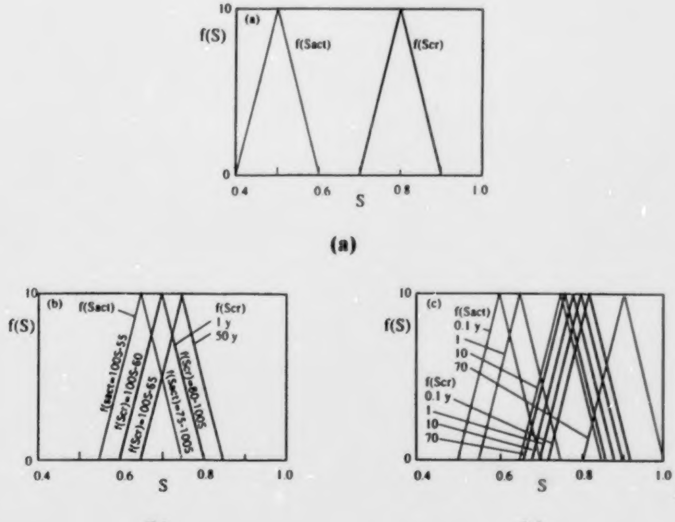
This equation requires that the lowest Sact-value of the function f(Sact) is lower than the lowest Scr-value of the function f(Scr). When the opposite is valid the probability of frost damage can be expressed:

$$P\{Scr<Sact\} = 1 - P\{Sact<Scr\} = 1 - \int_0^1 F_{Scr}\, f_{Sact}\, dS \tag{5}$$

Thus, the probability of frost damage can be calculated when the two functions f_{Scr} and f_{Sact} are known. As said above, both functions can be determined experimentally.

4 Service life with regard to frost attack

The service life will depend on the functions f_{Scr} and f_{Sact}. A number of cases can be discerned. Three of these will be treated below. In all three cases the frequency functions f_{Scr} and f_{Sact} are assumed to be triangular with the mean values Scr, m and Sact, m and the standard deviations σcr and σact which are both 0.04 (thus, the triangular distributions have the height 10 and the width 0.20).



(a)

(b) **(c)**

Fig. 3: Frequency functions of Scr and Sact
(a) Case 1: No time dependency of f_{Sact} and f_{Scr}.
(b) Case 2: Time-independent f_{Sact}. Time-dependent f_{Scr}.
(c) Case 3: Time-dependent f_{Sact} and F_{Scr}.

Case 1: No time-dependency of the functions f_{Scr} and f_{Sact}; (Fig 3a):

- Scr is independent of time. The mean value, Scr,m=0.80. The lowest value, Scr,min=0.70.
- Sact over a given period, such as one year, does not change with time. The mean value Sact,m=0.50. The highest value, Sact,max=0.60.

This means that all possible values of Sact are smaller than the lowest possible value of Scr. Therefore, the probability that frost damage will occur is zero irrespective of the exposure time. Therefore, the service life of the structure is not determined by frost damage but by some other deterioration mechanism.

A typical example where both Scr and Sact can be assumed to be independent of time is a brick façade. Brick is a material that hardly changes with time. Therefore, f_{Scr} is constant. Besides, a façade is a structural component that is normally exposed to about the same moisture load each year. Therefore f_{Sact} is constant.

Case 2: Time-independent function f_{Sact}. Time-dependent function f_{Scr}:(Fig 3b):

- Scr is gradually increasing with time. The mean value, Scr,m=0.70 after 1 year and Scr,m=0.75 after 50 years. Therefore, the distribution functions after 1 and 50 years are:

1 year:

$$0.60 \leq S \leq 0.70: \quad F_{Scr} = \int_{0.60}^{S} (100S-60)dS = 50S^2-60S + 18 \tag{6}$$

$$0.70 \leq S \leq 0.80: \quad F_{Scr} = 0.5 + \int_{0.70}^{S} (80-100)dS = -50S^2 + 80S-31 \tag{7}$$

50 years:

$$0.65 \leq S \leq 0.75: \quad F_{Scr} = \int_{0.65}^{S} (100S-65)dS = 50S^2-65S + 21.125 \tag{8}$$

- Sact does not change with time (the same as Case 1). The mean value, Sact,m= 0.65. Thus, the frequency function is:

$$0.60 \leq S \leq 0.65: \; f_{Sact} = (100S-55) \tag{9}$$

$$0.65 \leq S \leq 0.75: \; f_{Sact} = (75-100S) \tag{10}$$

$$S > 0.75: \quad f_{Sact} = 0 \tag{11}$$

Then, according to Eqn 4 the probability of frost damage after 50 years exposure time is:

$$P\{Scr < Sact\} = \int_{0.70}^{0.75} (50S^2-65S+21.125)(75-100S)dS = 0.042 \; \mathbf{(4.2\%)}$$

For 1 year exposure time the calculation is a bit more laborious and is not shown here. The calculations give a probability of frost damage after 1 year of **20%**. In this case the probability of frost damage decreases with increasing time as the critical moisture content increases with time. The most critical years are the first. This behaviour is valid for materials which change with time in a positive manner. A typical material is concrete which undergoes continued densification by hydration which probably leads to both an increase in Scr and a decrease in Sact. One example of the effect of time on Scr of concrete is shown in Fig. 2.

The opposite result is of course valid when the Scr-value decreases with time, from a mean value of 0.75 to 0.70, or if the Sact-value increases with time.

Case 3: Time-dependent functions f(Sact) and f(Scr); see Fig 3(c)

- Scr gradually decreases with time. The time function of the mean value is described by:

  $$\mathrm{Scr,m}= 0.80-0.02\log t \quad \text{(t in years)} \qquad \textbf{(12)}$$

  Then, the following mean values are valid:

  0.1 years: Scr,m=0.82
  1 year: Scr,m=0.80
  10 years: Scr,m=0.78
  70 years: Scr,m=0.76

  The shape of the function f_{Scrm} supposed to be unchanged.

- Sact increases with time. The following relation is used for the time dependency of the mean value, Eqn 2:

  $$\mathrm{Sact,m}= 0.55+0.10t^{0.3} \quad \text{(t in years)} \qquad \textbf{(13)}$$

  Then, the following mean values are valid:

  0.1 years: Sact,m=0.60
  1 year: Sact,m=0.65
  10 years: Sact,m=0.75
  70 years: Sact,m=0.90

The probability of frost damage is now calculated by Eqn 4 and Eqn 5. The following probabilities are obtained:

0.1 years:	P{Scr<Sact}=0	(0%, frost damage cannot occur)
1 year:	P{Scr<Sact}=0.0026	(0.3%)
10 years:	P{Scr<Sact}=0.308	(31%)
70 years:	P{Scr<Sact}=0.984	(98%)

Thus, the risk of frost damage increases with increasing time of water exposure. This case is valid for structures that are constantly exposed to liquid water. Typical examples are concrete dam walls or foundations in water. In normal cases, the Scr-value will not decrease with time but is kept constant. In such cases a less drastic reduction in the "probability of survival" than that calculated value above will occur.

5 Conclusions

The analysis in this paper shows that the frost resistance problem is of a stochastic nature. It also shows that no statement can be made concerning the service life with regard to frost resistance of a given structure in a given environment unless it is possible to transform the outer environment into an internal moisture-time field in all parts of the structure. This moisture field is of a highly stochastic nature.

The analysis also shows that the frost damage problem is of a nature that differs from many other destruction types in the sense that there is normally no well-defined time axis involved. Normally, for most destruction types, an increased exposure time causes increased damage until a certain limit is reached at which service life is ended. In the frost damage problem, an increased exposure time does not necessarily imply an increased risk of damage. Single freezings with a moisture content that is higher than the critical is enough to cause severe damage. But the occurrence of this dangerous situation is not necessarily a function of the exposure time. Frost damage might just as well occur during the first year as during the fiftieth, simply because the risk that the critical moisture content is trangressed is just as high during the first year, or even higher for materials that "mature" in a positive manner, like cement-bound materials. In many respects, the frost damage problem is similar to the problem of structural stability; failure occurs once the load-carrying capacity is transgressed. This might happen to structure, new as well as old one.

The analysis also shows that the uncertainty in a service life prediction with regard to frost attack is very large, which depends on the constable uncertainties in the internal moisture conditions that are reached inside the structure. The only rational way to ensure that the service life is high is to design the pore structure of the material in such a way that the probability that the critical moisture level will be transgressed during the real exposure is very low. This is done by designing the coarse pore-structure in such a way that it cannot absorb water easily (Fagerlund 1996). For a material like concrete this is achieved by entrainment of artificial so called air-pores. For brick it is achieved by burning the brick at a sufficiently high temperature. For other materials there are similar possibilities.

6 References

Fagerlund, G. (1972) *Critical degrees of saturation at freezing of porous and brittle materials.* Division of Building Technology, Lund Institute of Technology, Report 34 (In Swedish)

Fagerlund, G. (1979) Prediction of the service life of concrete exposed to frost action. In, *Studies on Concrete Technology*, Swedish Cement and Concrete Research Institute, Stockholm.

Fagerlund, G. (1992) Effect of the freezing rate on on the frost resistance of concrete. In, *Nordic Concrete Research*, Publ. No 11, Oslo.

Fagerlund, G. (1996) Predicting the service life of concrete exposed to frost action through a modelling of the water absorption process in the air-pore system. In, *The Modelling of Microstructure and its Potential for Studying Transport Properties and Durability*, (eds Jennings, H et al), Kluwer Academic Publishers.

Fagerlund, G. and Hedenblad, G. (1993) *Calculation of the moisture-time fields in concrete*. Division of Building Materials, Lund Institute of Technology, Report TVBM-3052.

Janz, M. (1997) *Methods of measuring the moisture diffusivity at high moisture levels*. Division of Building Materials, Lund Institute of Technology, Report TVBM-3076.

Klamrowski, G and Neustupny, P (1984) *Untersuchungen zur PrŸfung von Beton auf Frostwiderstand*. Bundesanstalt fŸr MaterialprŸfung, Forschungsbericht 100.

Powers, T.C. (1949) The air requirement of frost resistant concrete. *Highway Research Board, Proceedings*, vol 29.

Powers, T.C. and Helmuth, R.A. (1953) Theory of volume changes in hardened portland cement paste, *Highway Research Board, Proceedings*, Vol 32.

RILEM (1977) The critical degree of saturation method of assessing the freeze/thaw resistance of concrete. *Materials and Structures*, Vol 10, No 58.

Rombén, L. (1973) *Test results of capillary suction of cement mortar*. In, Report on Activities. The Swedish Cement and Concrete Research Institute, Stockholm.

119 MEDIC - A METHOD FOR PREDICTING RESIDUAL SERVICE LIFE AND REFURBISHMENT INVESTMENT BUDGETS

F. Flourentzou,[1] E. Brandt,[2] and C. Wetzel[3]

[1]Ecole Polytechnique Federale de Lausanne, Switzerland; [2]Danish Building Research Institute, Hoersholm, Denmark; [3]Fraunhofer Institut für Bauphysik, Holzkirchen, Germany

Abstract

The service life of buildings is an important factor e.g. in life cycle assessment and the assessment of global costs. Based on experience much information is available regarding the service life of building elements. However, for existing buildings such information is of little use as the key question is the probable date of repair/replacement. MEDIC is developed on the theories of conditional probabilities to help assess the residual service life and thereby the necessary investments in refurbishment.

When passing from working on general products, like the life span of wooden windows, to specific objects, for example 29-year old wooden windows, the current condition of the object must be taken into account. An evaluation of residual service life must for that reason be closely connected to a good diagnosis method. In the European project EPIQR on deterioration of building materials and components is described by the use of a classification system with four classes for the qualitative condition (e.g. of a facade or window). MEDIC calculates the probability to change from one class to another during time. The prediction is based on the combination of the a priori probability based on experience from a large number of previous investigations/refurbishments and the current state of the object under study.

Keywords: Budgets, building assessment, refurbishment investment, residual service life, service life prediction, probabilistic methods.

1 Introduction

When elaborating a refurbishment scenario, architects and engineers are interested in the residual service life of the building materials and components. When energy saving is one of the goals of the refurbishment operation, this information

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becomes of primary importance. A life cycle energy balance or a life cycle assessment of an existing element cannot be performed without an estimation of the remaining service life. Most data found in literature concerns new items which is of little use for existent and deteriorated building elements.

Knowledge of the probable residual life span of a building element will often be decisive for whether it will be replaced or not. For example, it is questionnable whether windows in a relatively acceptable deterioration state should be replaced only because they are energy inefficient. Knowing that scaffolding will anyway be installed for facade renovation, a replacement of the windows becomes more plausible to consider. To judge whether a replacement of the windows is energy and cost efficient can only be verified by a cost benefit analysis where the energy savings and the probable residual service life are taken into account. Obviously the analysis is highly dependent on whether the residual service life is assessed to 5 - 8 years or 25 - 30 years.

Consequently it is of importance to develop a model which can simulate the probable deterioration of all building elements thereby making it possible to determine their probable date of replacement. Knowledge of this deterioration for all building elements makes it possible to assess the global development in maintenance and refurbishment costs for the entire building. It also allows planning of future refurbishment actions by listing the probable elements to be replaced in 5, 10 or 15 years from the time of the assessment.

2 Existing methods

Quite a few methods are already in use (e.g. Genre J-L (1995) and Mayer P. et al., (1995)). Of special interest has been the work done for the *"Swiss Federal Office for Economic Policy"* in the framework of the PI BAT program: MEBI (Genre, 1995). MEBI bases its predictions on a large-scale survey of 30 building experts. It is capable of calculating the residual service life of 50 different building elements and assessing the development of the refurbishment investment budget under condition that no refurbishment work is undertaken in the meantime.

(Mayer et al, 1995) present the deterioration curves of 10 envelope elements studied on 120 buildings. This method distinguishes between three deterioration curves: L_{max} for elements in favourable conditions (good element quality, protected position, good maintenance, etc.), $L_{\emptyset}$ for normal conditions and L_{min} for unfavourable conditions (inferior quality, exposed to harsh weather, bad maintenance, etc.). For the element "cladding" for example, see figure 1, L_{max} corresponds to traditional cladding, in a non polluted environment and protected by an overhang. L_{min} corresponds to exposed synthetic cladding while $L_{\emptyset}$ corresponds to modern mineral claddings. The MEBI curve approximately corresponds to $L_{\emptyset}$ on the graph in fig: 1.

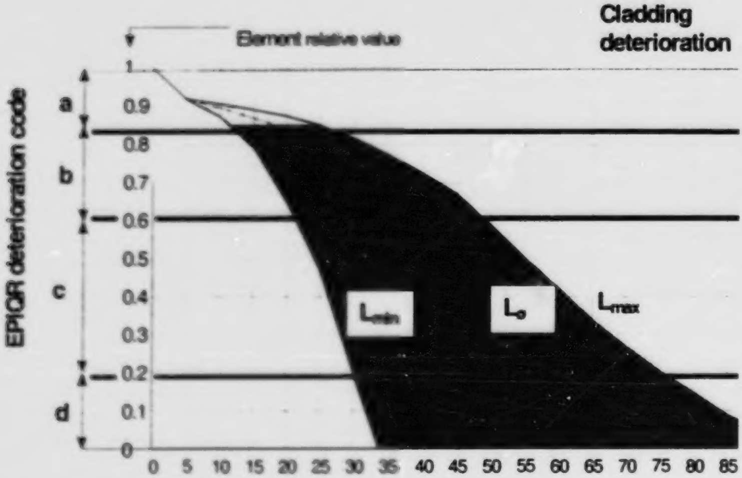


Fig: 1. Deterioration curves according to (Mayer P. et al.)

3 The MEDIC method

MÉDIC stands for "Méthode d'Évaluation de scénarios de Dégradation probables dÍnvestissements Correspondants" and is a proposal for a new method to be used together with a surveying tool which uses a classification of the degradation states of the elements, such as. EPIQR (Brandt et al. 1998). MÉDIC is intended for use with EPIQR and is based on sub-dividing the building into 50 elements. Four codes {*a*, *b*, *c*, *d*}, identical to those of EPIQR, are used to describe the deterioration state of the elements. Code *a* represents an element in good condition, code *b* an element with minor deterioration, code *c* an element with more serious deterioration and code *d* an element that needs replacement. The knowledge base of the method is summarised in 4 probability curves for each building element. For a certain element these curves show at any time in the elements' life time the probability for the deterioration code to be *a*, *b*, *c* or *d* respectively.

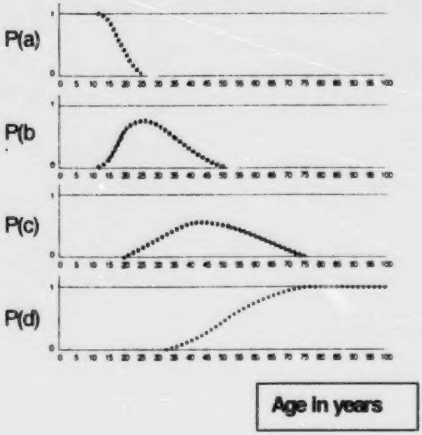


Fig: 2: Illustrative probability curves for deterioration codes of the element "cladding".

The probabilities of figure 2 can also be plotted on a cumulative curve, see figure 3. The sum of the probabilities shall of course at every instant be 1.

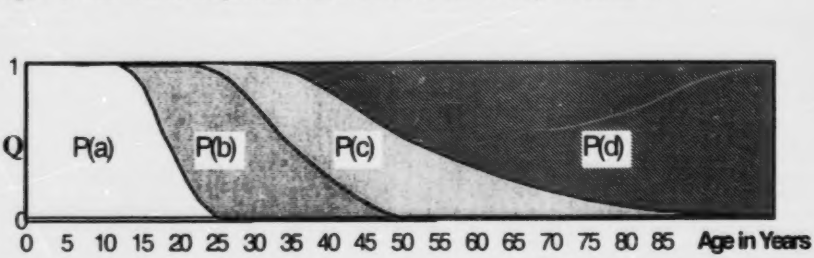


Fig: 3: Cumulative probabilities of deterioration codes

3.1 Examples

According to figure 3, a 10-year old cladding is certain to correspond to code *a*, while a 17-year old cladding is code *b* with 50% probability. A 40-year old cladding has a 35 % probability of being code *b*, a 40% probability of being code *c* and 15% probability of being code *d*. A 95-year old cladding will, with a 99% probability, need replacement.

3.2 Building quality space

On the y-axis of the graph in figure 3 the probability of a given building element to be code *a, b, c* or *d* respectively can be seen. The space Q{0, 1}on the y-axis represents the universe of all building elements in the whole building. Any specific element, q, is situated somewhere between 0 and 1 on the y-axis of the graph dependent on its quality. If we accept the hypothesis that element deterioration follows an affine curve to L_{min}, L_{max} or $L_{\emptyset}$ in figure 1, and if we know the exact position of an element on the y-axis of the graph, we can read the time of passage from one code to another. If q is a specific element, $q\in Q$, the value q=0 represents a good building with a long life span as all transitions from one code to another are taking place after very long time. It corresponds to L_{max} in figure 1. An element corresponding to the value q=1 in contrast represents the worst case. It corresponds to L_{min} in figure 1. The value q=0.5 corresponds to $L_{\emptyset}$. We call the space $Q=\{0,1\}$ the "Quality Space". In practice it is impossible to determine the position of a specific element exactly in the space Q, but it is possible to define or assess a region in which it is likely to be found. An element of poor quality for example belongs to $q\in\{0.5,1\}$ while an element of good quality belongs to $q\in\{0,0.5\}$.

3.3 Conditional probabilities

If we know that $q\in\{0, B\}$, conditional probabilities give

$$P(\text{ code i} \mid q\in\{0, B\}) = P(\text{code i}) / 0B. \qquad (1)$$

If $q\in\{B, 1\}$ (1) becomes:

$$P(\text{ code i} \mid q\in\{B, 1\}) = (P(\text{code i})-0B) / (1-0B). \qquad (2)$$

$i\in\{a, b, c, d\}$.

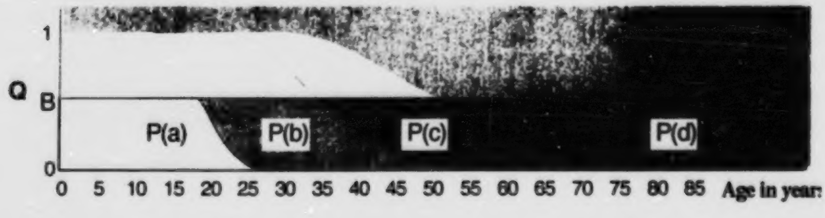


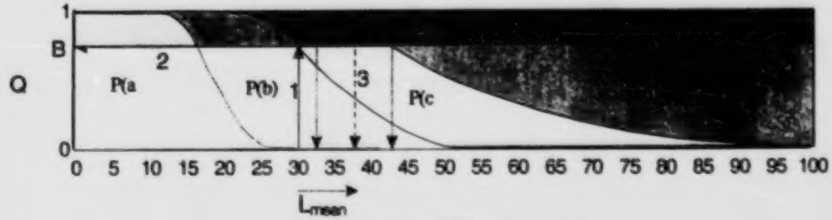
Fig. 4: If we know that a building is found in a subspace $q=\{0, B\}$ then $P(\text{code i} \mid q) = P(\text{code i}) / 0B$

As we can see from figure 4, the interval of probable transition from code *a* to code *b* is from the 12th to the 26th year, i.e. an interval of 14 years. Knowing that $q\in\{0, B\}$ restrains the time of transition to the interval from the 18th to the 26th year i.e. an interval of 8 years. The new probable period for transition from *b* to *c* is from the 34th to the 50th year and the new probable period of transition from *c* to *d* is from the 50th to the 95th year. As it appears, the knowledge of $q\in\{0, B\}$ considerably restrains dispersion on the probable residual service life.

3.4 Determination of the probable residual service life

During the building survey - EPIQR diagnosis - the surveyor determines the deterioration code of the 50 building elements. Further, information about the building age is available from the general information about the building. Combining the information about degradation code and age, it is possible to determine a sub-space q to which the element belongs, see figure 5. To do this, a line, arrow 1, is drawn from the element age to intersect with the curve between the area of the elements degradation code to the neighbouring area(s). Then draw a line, arrow 2, from the intersection point to the y-axis to determine point B. For instance, as shown in the figure for degradation code *c*, the arrow is intersecting the curve between areas for code *b* and code *c*. The quality subspace becomes q ={B, 1}, see figure 5, as the deterioration code has changed from *b* to *c*. (In case there are two intersections two vertical lines are drawn and the sub-space is determined as the "distance" from the lower to the higher value of y).

As we have chosen a probabilistic approach, the probable residual service life, L, is not a single figure but corresponds to an interval. To find the probable residual service life the point where the probability of having code *d* is 50% is determined, i.e. where y = (1+B)/2 intersects the next transition curve (the curve that separates P(*c*) and P(*d*)). A line, arrow 3, is drawn vertically from this point to intersect with the x-axis. The difference between this point and the age of the element give the probable residual service life L_{mean}. On the curve the upper and lower limits of the probable interval of L are determined as the intersections of the curve with y = B and Y = 1 respectively.



Residual service life, L = 7 ± 5 year

Fig. 5: Residual service life for a 30 year old cladding with code c

3.4.1 Example

The cladding of a 30 year old building has been allocated code *c*. The quality sub-space becomes q = {B, 1}, see figure 5. This means that the cladding is of bad quality. On the graph in figure 5 it can be read that at time t = 35 years, i.e. five years after the diagnosis date, P(at least *c*, t = 30) = 0.94. Also it can be seen that 0B = 0.76. Using equation **(2)**:

P(code *c*, t=35| code *c* at t=30) = (P(at least *c*, t=30)-0B) / (1-0B) = (0.94 - 0.76) / (1 - 0.76) = 0.75.

The a priori probable deterioration code of a 30-year-old cladding in 5 years from now is 0% code *a*, 49% code *b*, 45% code *c*, 6% code *d*. Knowing that today, 30 years after construction, the cladding is at code *c*, the probability for code *c* in 5 years from now becomes 75% and for code *d* 25% (codes *a* and *b* are excluded).

3.5 Evolution of refurbishment investment budget

After determining the present state and the quality subspace of an element, the a posteriori probability of having code *a*, *b*, *c*, or *d* for the 15 years that follow the diagnosis can be determined.

With a number of random draws on the probability curves for the 50 elements (Monte-Carlo procedure) and calculating the cost for the resulting code combinations, we can obtain the probability distribution of the global refurbishment cost.

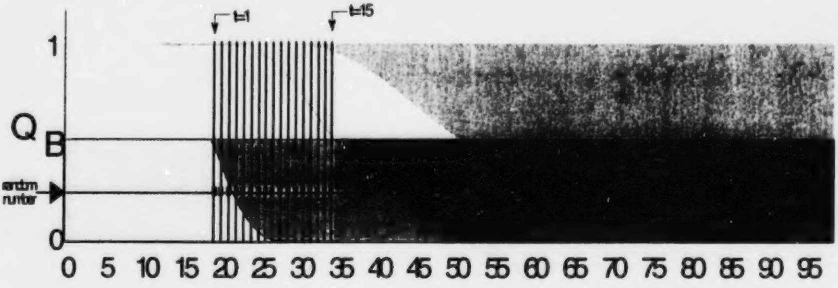


Fig. 6: Random number generation of the deterioration codes for the 15 years following diagnosis

Every random number gives a deterioration code for each year for an element. With the costs corresponding to the codes, we calculate the global refurbishment cost of a random building situated in the sub-space {q1, q2, q3, ….q50} 0 {Q1, Q2, Q3, …..Q50}.

If a sufficient number of random buildings situated in the building quality sub-space are drawn, the probability distribution of their refurbishment cost will approach the real probability distribution. Also the mean and the standard deviation of the cost distribution can be calculated. The minimum global cost on the curve can be interpreted as the refurbishment cost of elements with a long service life, i.e. good quality materials, protected location etc.

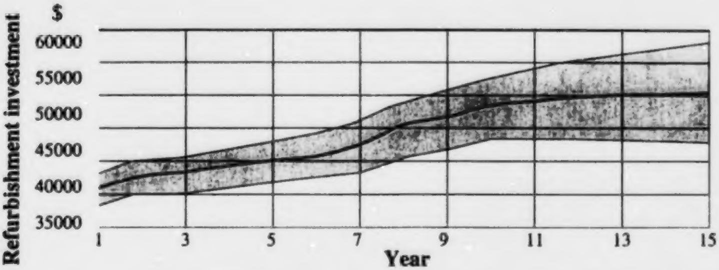


Fig. 7: Illustrative example of an expected curve of the evolution of the refurbishment investment budget

4 Conclusions

The residual service life of building elements is an important piece of information for refurbishment decisions that should be economically as well as sustainable. In order to determine residual service life correctly the current deterioration state of the elements must be taken into account. It is also necessary to adopt a system with distinct classes to describe deterioration, and the classes should have a physical meaning which can be observed and distinguished in practice. The EPIQR qualitative deterioration codes *a, b, c, d* is an example of a tool which offers a good basis for the modelling of the degradation over time.

MEDIC calculates the residual service life of a building element not as a deterministic unique value but as a probability distribution. To do so it combines a priori information collected on a large number of buildings with specific information on the examined building in order to calculate an a posteriori probability distribution with less dispersion.

The residual service life of building elements is not only used as a decision criterion in refurbishment scenarios but is also an important factor in Life Cycle Analysis. The evolution of the global refurbishment cost of the building is valuable information for investment planning. It can help the owner of a single building to decide which is the most judicious moment to undertake refurbishment, or the owner of a large building stock to organise refurbishment operations and determine his short and long term financial needs.

5 References

Genre J-L (1995), MEBI - Méthode d'Evaluation de Budgets d'Investissements , LESO-PB, EPFL, Lausanne.

Mayer P. et al., (1995), Vieillissement des éléments de construction et coûts d'entretien, Swiss Federal Office for Economic Policy, Program PI BAT, Zurich

Mayer Paul, Kurt Christen, "Financial Optimisation of the Time Interval between Renovation of a Building", ETH Zurich (to be published).

Brandt, E., Wittchen, K.B., A. Faist, Genre, J-L. (1998), EPIQR – A new surveying tool for maintenance and refurbishment, Proceedings 8th Conference on Durability of Building Materials and Components, Vancouver, Canada (to be published).

120 DEVELOPMENT OF A RELIABILITY-BASED DURABILITY DESIGN METHOD FOR TIMBER CONSTRUCTION

G.C. Foliente,[1] R.H. Leicester,[1] I. Cole,[1] and C. Mackenzie[2]

[1]CSIRO, Melbourne and [2]Timber Research and Development Advisory Council, Brisbane, Australia

Abstract

Formal durability design procedures for timber construction, based on prediction models and concepts similar to structural design, are under development in Australia. This paper presents an overview of key results of the sub-programs that comprise this major multi-disciplinary research effort; these are: (1) model development and calibration; (2) environmental agents – maps and databases; and (3) test protocols and material resistance. The ultimate goal is to develop a durability modeling and design software that will allow designers to predict the life expectancy of timber in certain applications, and make informed decisions on important variables that affect timber durability and building performance.

Keywords: timber/wood construction, durability design, modeling

1 Introduction

Durability is one of the most important considerations in the use of wood in construction. While many timber durability guidelines and design aids are available to engineers and designers (e.g., Sagot 1995, NAFI 1989, Dost and Botsai 1990), the current approach to timber durability design is still very much an art; design solutions vary from person to person and control of performance depend almost exclusively on compliance to good building practice. This tends to inhibit innovation and optimisation of building design, and is not useful for assessing the impact of rapid changes enforced by legislation (e.g., banning of chemicals).

If the factors affecting durability and the mechanisms of degradation, including inherent uncertainties, are quantified, design procedures can be developed so that a product or building can be designed to perform satisfactorily with a given level of confidence, much like the way it is currently done in structural design. This idea was

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first proposed by Leicester and Barnacle (1990) and investigated further by Stringer (1993). But while the technology of structural engineering is well developed and applied with confidence in design, the technology required for an engineering approach to durability is far behind.

This paper presents an overview and some preliminary results of a major multidisciplinary timber durability research effort in Australia, focused on the development of a reliability-based durability design method for timber construction. One factor that helps us appreciate the complexity of this undertaking is the fact that since the inception of the project, about 900 related references have been collected and reviewed (Leicester 1998).

2 Project description

2.1 Background and objectives

The timber industry in Australia identified durability as a high priority research need in early 1994. After further discussions with researchers, industry representatives, regulators and end users (Foliente 1995), an integrated research proposal to the Forestry and Wood Products Research and Development Corporation (FWPRDC) was submitted by five research organisations and was subsequently jointly funded as a three-year collaborative project for about AUD$1 million per year.

The long-term goal is to develop tools and methods to enable designers to: (1) predict the durability of any type of timber construction located anywhere in Australia; (2) make informed decisions on important variables affecting timber durability and their influence on performance; and (3) design and specify a durable system for any use or application that will achieve its performance goals.

The research program described herein is the first phase of an envisaged two-phase durability project. The primary objective in the first phase is the development of probabilistic durability prediction models and of a reliability-based design procedure for timber construction. In the second phase, the models will be further calibrated and validated, the design method will be trialed and refined, a durability design software will be developed, and other practical results and recommendations of the first phase of research will be implemented (e.g., test protocols and hazard maps will be proposed to standards committees).

The Timber Research and Development Advisory Council (TRADAC) coordinates the project and chairs the Project Technical Committee, which consists of sub-program project leaders and key researchers, and industry representatives. Research is divided into three inter-related sub-programs, which are described next.

2.2 Sub-programs

2.2.1 Sub-program 1: Model development and calibration

Research covering the development of degradation attack models and formalised durability design methods, is led by Dr R.H. Leicester and undertaken at the CSIRO Building, Construction and Engineering (CSIRO-BCE). The basic approach is introduced below; key aspects are described in the next section and in greater detail in Leicester and Foliente (1999).

Figure 1 illustrates the key steps in the development of the attack models and the design procedure. In the case of decay, for example, we start with classical models of moisture movement in porous media, using fundamental material parameters. Climate, soil and other environmental data are used to identify the context of durability performance, and as input to predicting performance. A simplified degradation or attack model, with new material parameter inputs that can be easily and quickly measured and with reasonable prediction capability (compared to the fundamental model), is then developed and calibrated using all available data including historical durability index properties, new field and laboratory data generated in the present project (e.g., from Sub-programs 2 and 3) and, when necessary, expert opinion. Then, the durability design procedure is developed from a composite of all the attack models.

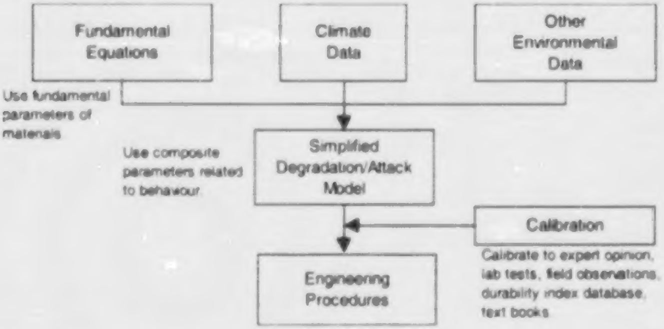

Fundamental Equations
Climate Data
Other Environmental Data
Use fundamental parameters of materials
Use composite parameters related to behaviour
Simplified Degradation/Attack Model
Calibration
Calibrate to expert opinion, lab tests, field observations, durability index database, text books
Engineering Procedures

Fig. 1: Schematic illustration of the development of the prediction model and the engineering design procedure

2.2.2 Sub-program 2: Environmental agents – maps and databases

This sub-program is led by Dr I. Cole (CSIRO-BCE), with participating researchers from the CSIRO Forestry and Forest Products (CSIRO-FFP), and Queensland Forest Research Institute (QFRI). This includes determining the levels of risk of various hazards and developing hazard maps for decay (in and above ground), termites and corrosion based on climate data and results of Sub-program 3 (Cole 1996; Thornton 1996).

Detailed climate data have been collected and are being related to microclimate within the house, and eventually to local element condition (Figs. 2 and 3). This process requires climate sub-models that relate global climate data to a specific site, from external climate to microclimate within the house, and from microclimate in the cavity to moisture conditions of the timber (Fig. 2). Climate data that have been collected include rainfall, temperature, relative humidity, wind, evaporation, cloud cover and various components of solar radiation. Microclimate data being monitored in cavities of selected houses in locations with tropical, subtropical and temperate climates are temperature and relative humidity (Fig. 3); also being monitored are material response parameters including time of wetness and moisture content of selected timbers. Conditions that are conducive to degradation (initiation and/or acceleration) are being investigated.

Based on seven years of Australian data on exposed L-joint specimens and the mean monthly temperature and rainfall in Australia, an above ground decay hazard map for Australia has been plotted (Fig. 4a) using the Scheffer's index (Scheffer 1971; Carter et al. 1993). Knowing that faster evaporation in the Australian climate allows wetted timber to dry out faster than in the northern hemisphere (where the Scheffer's index was originally derived), a new decay index, referred to as the DPI index, has been derived as a function of percentage of wet days and the average vapour pressure deficit (M. Cause, private communication). This index fits the updated nine-year L-joint data better than the Scheffer's index did. The differences are seen in comparing Figs. 4a and 4b; the DPI index provides a more refined and appropriate above ground decay hazard map for Australia. Further refinements in the calculation of the above ground durability hazard are being investigated.

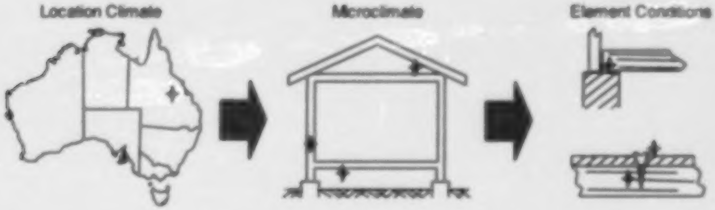


Fig. 2: Durability prediction from climate data to local elements

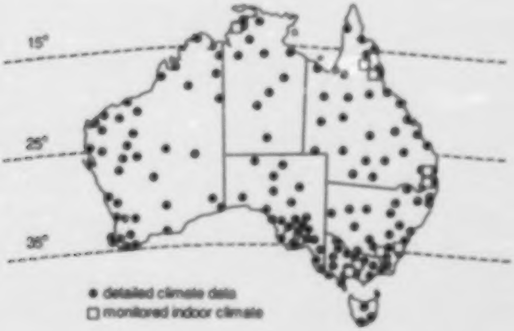
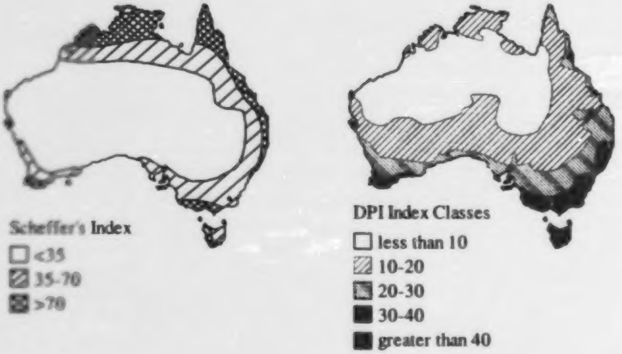


Fig. 3: Site locations for detailed climate data and for monitored indoor climate


Scheffer's Index
☐ <35
▨ 35-70
▩ >70
DPI Index Classes
☐ less than 10
▨ 10-20
▩ 20-30
■ 30-40
■ greater than 40

(a) **(b)**

Fig. 4: Above ground decay hazard map: (a) Scheffer's index (after Carter et al. 1993); (b) DPI decay index (after M. Cause, private communication)

The termite hazard map will be based on a combination of results of a public survey and prediction using the attack model. The first stage of a national survey has been completed and the second stage is on going at the time of this writing.

Metal corrosion is a combination of atmospheric corrosion on exposed steel and cathodic corrosion on the embedded steel. For the case of atmospheric corrosion, critical attack parameters are the effective time of wetness and concentration of air pollutants, particularly sea-salt. Based on analysis of available exposure test data and modeling results, Fig. 5 shows the estimated corrosion of a metal plate of mild steel containing copper.

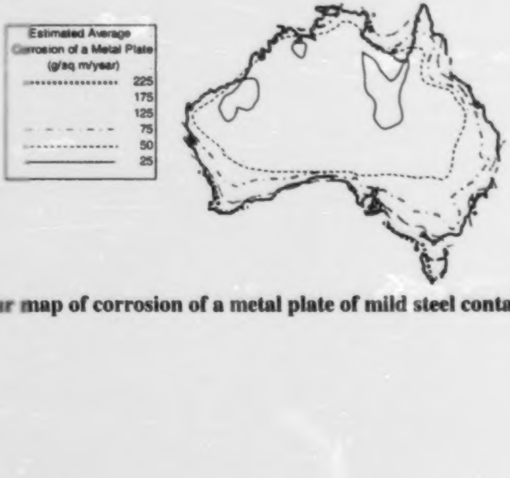

Estimated Average Corrosion of a Metal Plate (g/sq m/year)
225
175
125
75
50
25

Fig. 5: Contour map of corrosion of a metal plate of mild steel containing copper

2.2.3 Sub-program 3: Material resistance and test protocols

This sub-program is led by Dr J Thornton (CSIRO-FFP), with researchers from CSIRO-FFP, QFRI and the State Forests of New South Wales (SF-NSW) also involved. Part of this sub-program is focussed on developing standardised durability assessment methods for materials, including the suitability of accelerated field simulation laboratory techniques to determine resistance to termites, fungi and corrosion (Thornton 1996). Another focus is the determination of durability rating resistance of common timber products for prescriptive design guidelines and of degradation rates of treated and untreated timber for prediction models. The latter is based on both new field data and previously collected data originally intended to obtain durability index properties.

A considerable amount of material resistance evaluation is in progress, with decay and termites, both in-ground and above ground, for both treated and untreated wood. There are also above ground wood panel exposure tests and corrosion tests with both nails and metal plates; see Thornton (1996) for details.

3 Model development

3.1 Basic concept and methodology

The key to developing a design procedure is first to have predictive models for durability. Figure 6 illustrates the way the models and design procedure will be used. A set of input parameters is used to define the design situation, a particular hazard type is selected, and a calculation model is used to predict the performance of the element or building system.

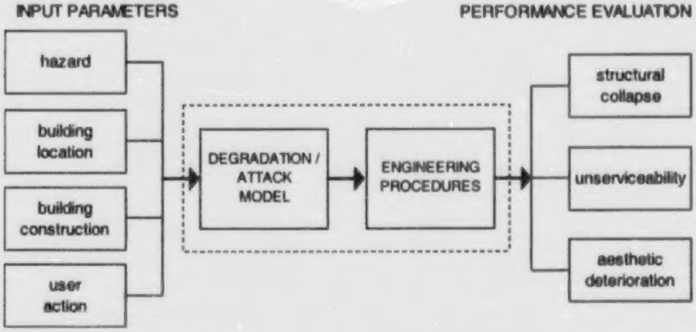

INPUT PARAMETERS
PERFORMANCE EVALUATION
hazard
building location
building construction
user action
DEGRADATION / ATTACK MODEL
ENGINEERING PROCEDURES
structural collapse
unserviceability
aesthetic deterioration

Fig. 6: Schematic illustration of the use of prediction models and engineering design procedures

Typical input parameters are those that relate to the hazard, building location, building construction, building element and maintenance programs (or user action). Performance criteria will be classified as either structural collapse, unserviceability (such as excessive deflections, water entry or loss of material) or aesthetic deterioration. The hazards considered include attack by:

- Fungi
- Termites
- Corrosion agents
- Mechanical degradation

Since there are uncertainties in the data, models and analysis methods, it is natural to apply probabilistic techniques and recognize and incorporate these uncertainties in the model development process. Each attack model is assumed to comprise a sequence of events as shown in Fig. 7. The duration of each event is a random variable. This leads to a prediction in statistical terms as shown in Fig. 8.

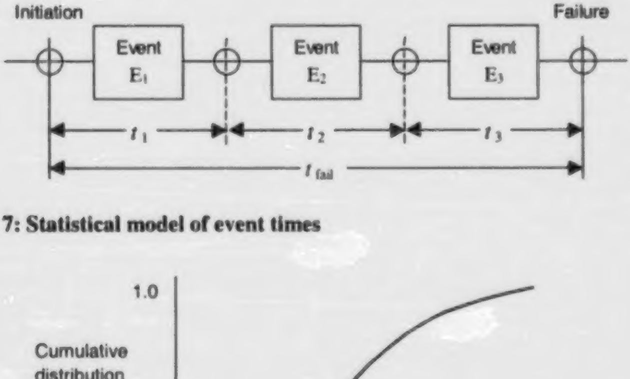


Fig. 7: Statistical model of event times

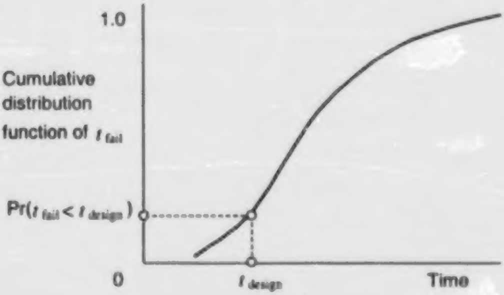


Fig. 8: Statistical representation of time to failure

The variability of the event times should take into consideration not only the natural in-service variability of these events, but also the additional variability that reflects the uncertainties arising from our inadequate knowledge or from the use of simplified composite models. Of particular interest is the risk parameter $\Pr(t_{\text{fail}} < t_{\text{design}})$, the probability that the time to failure t_{fail} is less than the specified design life of the structural element, t_{design} (Fig. 8) Typical acceptable values of the risk parameter are 0.05, 0.2 and 0.5 for structural collapse, unserviceability and aesthetic, respectively.

3.2 Attack models

The attack models being developed are primarily focused on decay, termite and corrosion. Details about the development of decay (in-ground and above ground) and termite attack models are given in Leicester and Foliente (1999). The former is based on the ability to predict the movement of free water in timber. The latter is based on surveys of the opinion of experts on termites. Details about the development of the corrosion model are discussed in Cole et al. (1999a, 1999b). These models are based on simple equations that require the measurement of only a limited set of parameters.

4 Durability design and other applications

4.1 Design

General prediction models are too complex to be used directly in design codes. Figure 9 shows an idealised diagram of how a general prediction model or an analysis based on "first principles" can be used to develop an engineering design procedure and eventually to develop simplified prescriptive design rules. Current methods have been formulated via Path A. The present work is primarily focused on the development of prediction models, which are calibrated using the available durability knowledge base (Path C) as described earlier. Ultimately, the models will be used to improve prescriptive durability design rules either directly or through an engineering design procedure (Paths E and D).

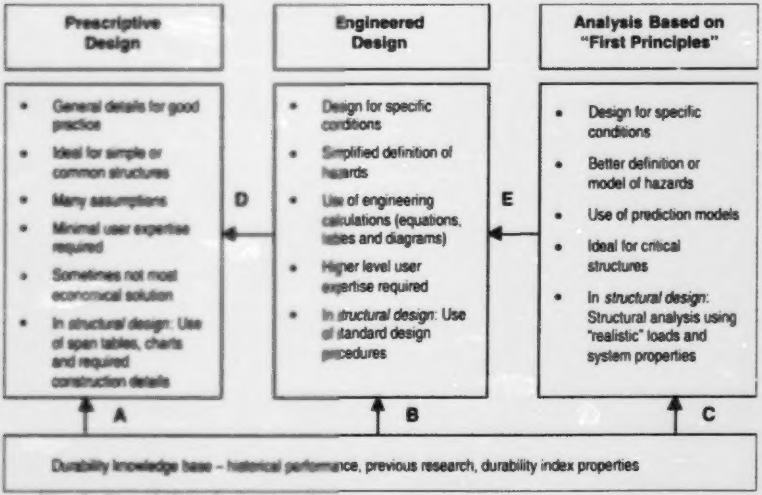

Prescriptive Design
Engineered Design
Analysis Based on "First Principles"
General details for good practice
Ideal for simple or common structures
Many assumptions
Minimal user expertise required
Sometimes not most economical solution
In *structural design*: Use of span tables, charts and required construction details
Design for specific conditions
Simplified definition of hazards
Use of engineering calculations (equations, tables and diagrams)
Higher level user expertise required
In *structural design*: Use of standard design procedures
Design for specific conditions
Better definition or model of hazards
Use of prediction models
Ideal for critical structures
In *structural design*: Structural analysis using "realistic" loads and system properties
D
E
A
B
C
Durability knowledge base – historical performance, previous research, durability index properties

Fig. 9: Durability design methods

In engineered design, the obvious practical approach would be to decompose the general model into a sequence of several simple sub-models. This would be analogous to the structural analysis of a complex structure by first replacing it with simpler analytical models.

For simplicity, input data should be in the form of a table or a map, such as the map for DPI decay index in Fig. 4b, or in the form of classifications based on evaluations according to some sort of protocol. Factors that should be considered in the design process include:

- location and orientation of the structure;
- structural detailing, particularly at joints;
- treatment and thickness of timber elements;
- use of coatings; and
- use of inspection and maintenance activities.

4.2 Other applications

The general prediction models can also be used to assess the performance of new and innovative products and the impact of rapid changes enforced by legislation (e.g., banning of chemicals). In conjunction with a non-destructive testing method, the models can be used to predict the residual life of a timber element and to develop an optimised inspection and maintenance program. Thus, the models are key components of an improved asset management system for timber structures.

5 Conclusions

This project provides a framework and method of using fundamental degradation models, and climate and relevant environmental factors to develop general durability prediction models and formal design procedures for timber construction.

Research to date indicates that the development of formal durability design procedures, based on prediction models, is not only feasible but is a better and more flexible approach to ensure durable timber construction. The existence of formal timber durability design procedures, similar in concept to structural design, will facilitate the teaching and globalisation of the technology, and hopefully lead to the day when failures related to durability will be regarded as failures of applied technology, and not taken to be an unknown risk associated with the use of timber.

Much research is needed to better understand fundamental degradation mechanisms of timber in-service, improve the attack models and provide more reliable data to validate and calibrate the prediction models.

6 Acknowledgment

The authors are grateful to FWPRDC and the Australian timber industry for partial funding of this project and the numerous participants in the project who have cooperated with us in model development and in the preparation of this paper.

7 References

Carter, J.K., Cause, M. and Moffat, A. (1993) A preliminary above-ground wood decay index map of Australia. *Proceedings of 24th Forest Products Research Conference*, 15–18 November, CSIRO, Clayton, Australia.

Cole, I.S. (1996) FWPRDC durability design project. Sub program 2 – Environmental hazards and mapping. *Proceedings of 25th Forest Products Research Conference*, 18–21 November, CSIRO, Clayton, Australia, Vol. 2, Paper 2/20, 8p.

Cole, I.S., King, G.A., Trinidad, G., Chan, W.Y. and Paterson, D. (1999a) An Australian wide map of corrosivity: A GIS approach. *Proceedings of 8th International Conference on Durability of Building Materials and Components,* 30 May–3 June, National Research Council of Canada, Ottawa, Canada.

Cole, I.S., Trinidad, G. and Chan, W.Y. (1999b) Prediction of the impact of the environment on to timber components: A GIS approach. *Proceedings of 8th International Conference on Durability of Building Materials and Components,* 30 May–3 June, National Research Council of Canada, Ottawa, Canada.

Dost, W.A. and Botsai, E.E. (1990). *Wood: Detailing for Performance*. GRDA Publications, First Edition, Mill Valley, CA, USA.

Foliente, G.C. (1995) Timber Durability Design Workshop – Proceedings. CSIRO Building, Construction and Engineering, Highett, Victoria.

Leicester, R.H. (1998) Prediction models for durability of timber construction: 0. Reference Master File. CSIRO Building, Construction and Engineering, DBCE Doc. 98/143(M), July, 82p.

Leicester, R.H. and Barnacle, J.E. (1990) Durability – A structural approach. *Proceedings of 23rd Forest Products Research Conference*, 19–22 November CSIRO Forestry and Forest Products, Clayton, Australia, Vol. 1, Paper 2/4, 9p.

Leicester, R.H. and Foliente, G.C. (1999) Models for timber decay and termite attack. *Proceedings of 8th International Conference on Durability of Building Materials and Components,* 30 May–3 June, National Research Council of Canada, Ottawa, Canada, 10p.

NAFI (1989) *Timber – Design for Durability*. National Association of Forest Industries, Timber Datafile P4, NAFI, Canberra, Australia, 31p.

Sagot, G. (1995) Detailing for durability. In *Timber Engineering STEP1*, (ed. H.J. Blass et al.) Centrum Hout, The Netherlands, Lecture A14, 9p.

Scheffer, T.C. (1971) A climate index for estimating potential for decay in wood structures above-ground. *For. Prod. J.*, Vol. 21(10), pp.25–31.

Stringer, G.R. (1993) The development of reliability based durability design methods for timber structures. Gottstein Fellowship Report, Gottstein Memorial Trust Fund, CSIRO Forestry and Forest Products, Clayton, Australia.

Thornton, J.D. (1996) FWPRDC durability design project. Sub program 3 – Test protocols and materials resistance. *Proceedings of 25th Forest Products Research Conference*, 18–21 November, CSIRO, Clayton, Australia, Vol. 2, Paper 2/22, 11p.

121 DURABILITY ASSESSMENT OF BUILDING SYSTEMS

J. Lair,[1] J. F. Le Teno,[1] and D. Boissier[2]

[1]Centre Scientifique et Technique du Bâtiment (CSTB), Grenoble, et
[2]Centre Universitaire des Sciences et des Techniques (LERMES/CUST),
Clermont-Ferrand, France

Abstract

Assessing the service life of building products is relevant for all building actors (insurers, manufacturers, building owners and architects). Indeed, the knowledge of building products service lives leads to a reduction of maintenance costs and environmental impact, and an improvement of safety. This paper deals with a methodological approach for durability assessment. The major steps are :

- Research of available durability data and their organization in a graph structure followed by the assessment of belief and plausibility distribution of service life.
- A Failure Mode and Effects Analysis, including a structural and a functional analysis in order to search all potential failures (weathering factors, product design and setting up).

The proposed method is a multi-model and multi-scale approach ; multi-model in order to adjust the model with our knowledge and our aim (modelling real life of building, but not a too complex and unusable model), multi-scale to take into account the links between the three geometric scales materials/products/building.

Finally, it gives (1) a distribution of nominal service life, for normal weathering processes, with corresponding belief and plausibility degrees, (2) details on the design and setting up problems, on exceptional weathering phenomena, which could lead to a shorter service life.

Keywords: belief, data quality, data sets, durability, FMEA, failure, functional analysis, hypergraph, nominal service life, plausibility, service life, structural analysis.

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1 Introduction

The forecasting of building durability is needed for economic, environmental and safety reasons. Durability can be defined as "the ability of a building and its parts to perform its required functions over a period of time and under the influence of agents". (British Standards Institution, 1992)

This paper deals with a methodological approach for building and building products durability assessment. But, instead of durability assessment with current methods (artificial ageing, models, fundamental studies in materials science) or with current data sets (field tracking studies and experts), our method is based on the use of all the available data.

The product durability will be defined by :

- the assessment of *nominal service lives* and *belief degrees*, which are the expected service life of the product under normal conditions and the belief we have in this assessment,
- an exhaustive list of the *potential failures*, according to design problems, setting up faults, exceptional use, maintenance errors (with FMEA).

2 Generalities

2.1 Data characteristics

The different sources of aging data are (Martin et al., 1996; Lair et al., 1998):

- artificial and natural tests ("short term laboratory based exposure") (Eurin et al., 1985),
- field tracking studies, in situ weathering, experts knowledge ("long term in service or outdoor exposures") (Ishizuka, 1983)
- fundamental mechanistic studies (Martin et al., 1996)
- reliability studies (Siemes et al.,1985).

According to the data sources, the data format is different. We propose a characteristic review of each data, including the scale of the study (system, sub-system, component), hypothesis, format of the result (quantitative deterministic, probabilistic or possibilistic data, qualitative orderly or non-orderly data), the type of test and the reference. As we will see further, we will associate a mass to each data, according to these characteristics and corresponding to the quality or credibility of the data. Unfortunately, these data are often incomplete, uncertain, and imprecise.

2.2 Evidence theory

To take this uncertainty into account, we propose to use the evidence theory (Shafer, 1976). Indeed, evidence theory and belief functions are tools of "modelling and quantifying the belief we can have in events which probability occurrence is unknown" (Bouchon-Meunier, 1995). According to Pearl, belief "consists of assertions about a specific situation inferred by applying generic knowledge to a set of evidence sentences." (Pearl, 1990)

Let's assume a set Θ, called *Frame of Discernment*, which is a finite set of alternatives (for example, when searching a product service life, Θ is the temporal scale $[0, t_{max}]$ with t_{max} arbitrary high bound). Let's associate to each subset T of Θ, a basic probability assignment *m(T)*, a belief *Bel(T)* and a plausible belief *Pl(T)*. m(T), Bel(T) and Pl(T) represent respectively the strength of an evidence, the reasons we have to believe T and the reasons we have to believe T if all unknown facts were to support T.

The relation between Bel(.) and m(.) is:

$$Bel(A) = \sum \{m(B) | B \subseteq A\} \tag{1}$$

Pl(.) satisfies the following formula (where ¬A is "Not A"):

$$Pl(A) = 1 - Bel(\neg A) \tag{2}$$

The use of belief functions result from assigning probabilities to sets rather than to individual points. With evidence theory, it is possible to model the narrowing of a hypothesis set with the accumulation of evidence.

3 Aim of the research

The aim of this research is the modelling of degradation phenomena of all materials, products and buildings, so that actors (insurers, manufacturers, building owners and architects) easily can have an opinion on the building system's behaviour. All the building actors are interested in the service life of a building system, whatever the degradation factors are. So, we have to model the "real" life of any building system while considering phenomena and actors as fixed parameters (Fig. 1).

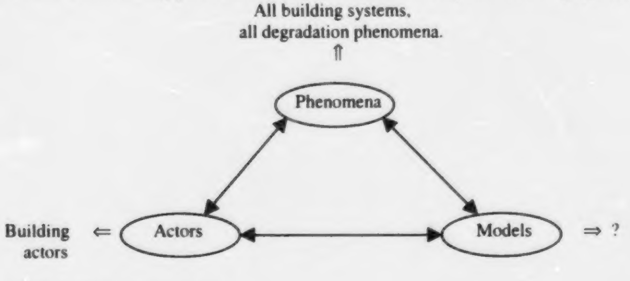

Fig. 1 : Modelling parameters

In our opinion, the building of a single model – modelling the behaviour of a system as well as the change in time of this behaviour, as structures and properties change – is a too complex task. Our proposal is a multi-model and multi-scale approach. For a multi-model approach, we build a normal functioning model, and some other models in case of structural change (for example, setting up faults, lower material quality…).

For a multi-scale approach, we take into account the decomposition of the system (building, products) into sub-systems (products) and components (materials). Each decomposition level seems to correspond to a different data source. For example (Lair et al. 1998), we have durability data of a roofing system with field tracking studies. One of the sub-system, the waterproofing layer, is well-known with artificial aging tests, and we assess the behaviour of its major component, the polymer bitumen, through its physical properties. Sometimes, it would be possible to model the behaviour of a new or unknown product combining the known behaviours of its components and their interactions.

4 Method

The three steps of the proposed method are (Lair et al. 1998):

- Failure Mode and Effects Analysis (FMEA),
- Data compilation and organization,
- Service life and belief degree assessment.

4.1 Failure mode and effects analysis (FMEA)

Used in electronic and aeronautic industries, the FMEA is a method of failure modes search. It gives an exhaustive list of all potential failures.

This approach requires:

- A structural analysis of the system in order to define the sub-systems and components, their characteristics, the physical, chemical and mechanical links;
- A functional analysis to understand the behaviour of the system under weathering and use factors (we have to know the system environment: degradation factors check-list);
- The FMEA, i.e. the experts opinion compilation.

To compile failure modes data, we propose to fill Table 1, a FMEA adapted to the building domain :

Table 1 : FMEA framework for a roofing system

Elements	Function	Modes	Solicitations Defects	Reasons	Inner Effects	Outer effects
Water proofing	Impermeability	Cracking	Loads	Wear and tear	Permeability	Moisture in insulation
	Impermeability	Boring	Punctual stress	Gravel	Permeability	Moisture in insulation
			Punctual stress	Maintenance load	Permeability	Moisture in insulation
				...		

The first column is filled with structural analysis data. Using functional analysis, we complete columns number 2, 3 and 4. Experts "imagine" columns 5, 6 and 7.

As we see, the degradation of a sub-system not necessarily lead to the failure of the system, but can involve a structural change and then an unexpected solicitation of a neighbouring sub-system (inner degradation but outer action change).

So, we determine not only sub-system failures, but also chaining events leading to the system failure.

Finally, it will be possible to determine the failure modes resulting from design errors (dimensional problems, material choices…) or setting up and maintenance faults (materials quality, setting up conditions…). For example, we have to notice that a faulty joining of plies in a roofing system (glue choice problem or too low temperature during setting up) will lead to a premature failure through a loss of water impermeability and moisture development in the insulation layer.

Therefore, we obtain a list of potential failure modes from a dialog with experts, in which the time is only a qualitative parameter (durability not expressed as service life but as ordinal classification).

4.2 Data compilation and organization

Now, we have to collect the available data, in any format, in any scale, on every phenomenon which may occur during the life of the system. We can also make analogies with similar products and/or similar environmental conditions. Let's notice that FMEA helps us in this search.

After that, we have to organize all these data. Assume we have several superposed graphs; the set is called *Durability hypergraph*. Each graph is a more or less precise picture of the system, built with a set of data. A graph is made of nodes (durability data and models) and links (existence of common parameters between two data). In top graphs, the temporal, geometric or phenomenological granularities are large (close to one). In bottom ones, they are small (close to zero). Granularity is defined as the ratio between the level of decomposition and the size of the system. It represents "grain fineness". The finer the grain is, the smaller the granularity is. When we have equivalent granularities, we consider that the data are on the same level.

As an example, consider a reinforced concrete wall, painted on its external face. The service lives are classified in a complete preorder (Table 2)

Table 2 : Available data and their corresponding granularities

Service life (year)	Level	Reference	Geometric granularity	Temporal granularity	Phenomenological granularity
20	1	[1]	product	Limit state	not explicit
150	1	[2]	product	Failure time	not explicit
> 30	1	[3]	product	failure time	not explicit
[25, 100]	1	[4]	product	failure time	not explicit
[100, 120]	2	[5]	material (RC)	failure time	not explicit
D(100,80)	2	[6]	RC slab	failure time	Carbonation
[5, 130]	3	[7]	steel, concrete, paint	failure time	not explicit
[55, 75]	4	[8]	phenomenological decomposition	limit state or failure time	according to the decomposition level

The references are:

[1] – (OPAC38, 1993). The limit state is maintenance interval against carbonation risk.

[2] – (EPFL-LESO, 1995)

[3] – (Perret, 1995)

[4] – (Gumpertz & Schwartz, 1996)

[5] – (Kürti, 1971)

[6] – (Siemes *et al.* 1985): D (100,80) is a probabilistic distribution with mean μ = 100 years and standard deviation σ = 80 years.

[7] – (Pihlajavaara 1980). We have three nodes: concrete, steel, paint. Their corresponding service lives are SL (concrete | very severe exposure) $\in$ [10, 100 years], SL (iron | very severe exposure) $\in$ [1, 10 years], and SL (paint | severe exposure) $\in$ [2, 20 years]. Reinforced concrete wall service life of the level is calculated with an appropriate algorithm.

[8] – This level is composed of several nodes, corresponding to degradation phenomena of reinforced concrete components, i.e. the concrete cover (carbonation, chloride penetration, sulfates, thawing) with and without cracking (let's notice again that some phenomena are strongly correlated), the steel (corrosion) and the paint (gloss, cracking). Several references as (Sarja 1994; Papadakis 1992)

4.3 Service lives and belief degrees

For each level (or graph), we assess the service life of the studied system from the durability data, with appropriate algorithms. To each service life, we assign a mass m (which is similar to a probability assignment) according to the quality of the basic durability data. Thus *m* is a quality indicator according to the origin of the data. Theoretical structures, data input, set-up conditions… are quality criteria (Funtowicz & Ravetz 1990) valued on a five-level scale, an example of which is given in Table 3. This idea will be developed further.

Table 3: pedigree criterion

Level	Criterion "Theoretical structures"
4	Established theory
3	Theoretically-based method
2	Computational model
1	Statistical processing
0	Definitions

Let's now assume that Θ is the finite temporal scale ($[0, t_{max}]$). Each durability data correspond to a *support function* on Θ. The degrees of support are easy to specify (in this case, the data "points precisely and unambiguously to a single non-empty subset A of Θ" (Shafer 1976)):

$$M(B) = \begin{cases} 0 & \text{if} \quad \text{B does not contain A} \\ m & \text{if} \quad \text{B contains A but } B \neq \Theta \\ 1 & \text{if} \quad B = \Theta \end{cases} \qquad (3)$$

We apply Dempster's rule (Shafer, 1976) to combine all support functions BEL_1, BEL_2 (focusing respectively on A_1, A_2, ...). The resulting support function is independent of the combination order; and is given by :

$$\mathrm{BEL(A)} = ((\ldots(\mathrm{BEL}_1(A_1) \oplus \mathrm{BEL}_2(A_2)) \ldots) \oplus \mathrm{BEL}_{n-1}(A_{n-1})) \oplus \mathrm{BEL}_n(A_n) \qquad (4)$$

However durability data are often conflicting data. In order to assess the validity of Dempster's combination, let's define $CON(BEL_1, \ldots, BEL_n)$ the *weight of conflict* (between evidences). The weight of conflict between BEL_1 and BEL_2 is the quantity:

$$CON(BEL_1, BEL_2) = \ln K = \ln \frac{1}{1-k} \quad with \quad k = \sum_{\substack{i,j \\ A_i \cap B_j = \emptyset}} m_1(A_i) m_2(B_j) \qquad (5)$$

We can now assess belief and plausibility degrees of service lives (Bouchon-Meunier, 1995). For example, considering the data A = [25, 100years] and its corresponding mass m([25, 100]years) ≠ 0; we have:

- If B = [0, 20[years then Bel(SL ∈ B) = 0 and Pl(SL ∈ B) = 0.
- If B = [0, 50[years then Bel(SL ∈ B) = 0 and Pl(SL ∈ B) ≠ 0.
- If B = [0, 110[years then Bel(SL ∈ B) ≠ 0 and Pl(SL ∈ B) ≠ 0.

The decision criterion is the confidence interval [Bel, Pl] (Dromigny-Badin et al., 1997). It expresses the knowledge we have about an assumption. The closer we are to [1, 1], more confident we are about our assumption; the closer we are to [0, 0], the less probable the assumption is; [0, 1] gives no information.

The methodological approach is shown in Fig. 2 : (1) compile available durability data, (2) organize data in a hypergraph, (3) assess service life and mass of each level, (4) combine them to obtain belief and plausibility of service life.

5 Results

Table 4 represents a non exhaustive list of available data. For the moment, we have used arbitrary masses m. Let's note that m(.) is an intrinsic value of the data. Thus, we could classify the data according to their own quality, and compare the belief and plausibility degrees of different products.

Table 4 : Data compilation and mass assignment

Level	Durability data	Mass m(.)
1_1	20 years	0.1
1_2	More than 30 years	0.7
1_3	[25, 100 years]	0.8
1_4	150 years	0.2
2_1	Distribution (μ=100, σ=80)	0.5
2_2	[100, 120 years]	0.6
3	[5, 130 years]	0.4
4	[55, 75 years]	0.6

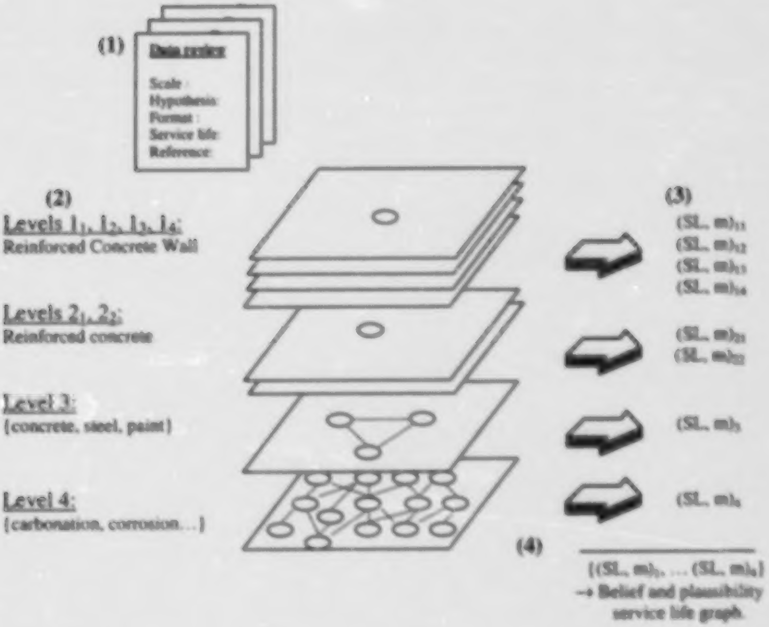
(1)
Data series
Scale :
Hypothesis:
Format :
Service life
Reference
(2)
Levels $1_1, 1_2, 1_3, 1_4$:
Reinforced Concrete Wall
Levels $2_1, 2_2$:
Reinforced concrete
Level 3:
{concrete, steel, paint}
Level 4:
{carbonation, corrosion…}
(3)
$(SL_i, m)_{11}$
$(SL_i, m)_{12}$
$(SL_i, m)_{13}$
$(SL_i, m)_{14}$
$(SL_i, m)_{21}$
$(SL_i, m)_{22}$
$(SL_i, m)_3$
$(SL_i, m)_4$
(4)
$\{(SL_i, m)_1, \dots (SL_i, m)_4\}$
→ Belief and plausibility service life graph

Fig. 2 : Methodological approach

As an example, assume we want to combine the two evidences: $A = \{SL / SL \in [5, 130 \text{ years}]\}$ and $B = \{SL / SL \in [55, 75 \text{ years}]\}$. Assume that their corresponding masses are 0.2 and 0.5. $A \cap B \neq 0$ so they are heterogeneous evidences. Dempster's rule involves: $M(A \cap B) = 0.1$; $M(A) = 0.1$; $m(B) = 0.4$ and $M(\Theta \setminus \{A, B\}) = 0.4$. Since we are in the particular case $B \subset A$, $M(B) = 0.4 + 0.1 = 0.5$ (the belief degree of a subset is the sum of its belief degree and the belief degrees of its subsets); so the resulting separable function will be: $M(A) = 0.1$; $M(B) = 0.5$; $M(\Theta \setminus \{A, B\}) = 0.4$.

Combining all the support functions, we obtain the resulting belief function. The Fig. 3 : Service lives belief and plausibility graph shows the service life belief and plausibility degrees for the reinforced concrete wall, given that $CON(BEL_1 \dots BEL_n) = 0.62$.

Given a Bel-Pl level, this figure shows that we can strongly believe (90%) that a [100, 130] years concrete wall structure would have failed. The reinforced concrete wall failure may occur in the interval [0,100] years with a less than 60% belief degree, given that only a 10% belief degree corresponds to [0,75] years, it is hardly believed that failure would occur before a 75 years time.

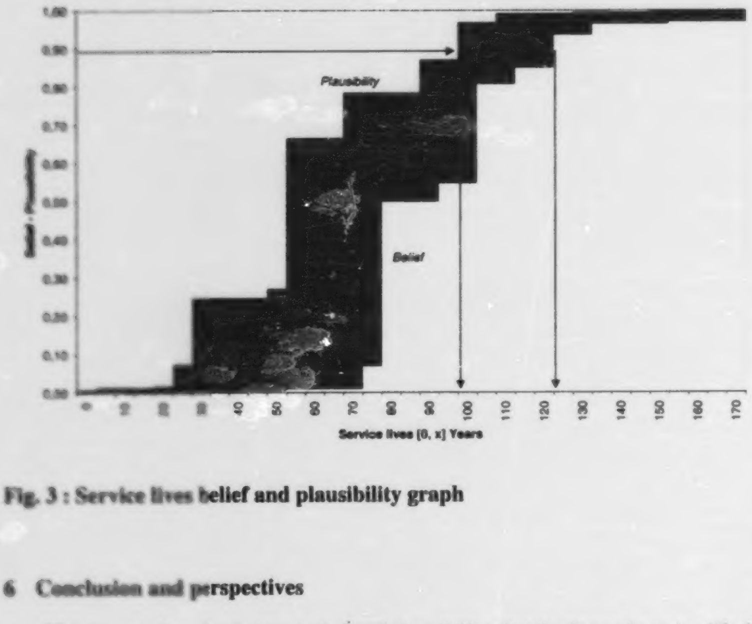

Fig. 3 : Service lives belief and plausibility graph

6 Conclusion and perspectives

The proposed method gives belief and plausibility distributions of service life for any building system (traditional or new systems), built on available durability data. Combined with FMEA analysis, we also have an exhaustive list of possible defects or faults (design, setting up, maintenance) leading to a premature failure of the studied system. This method handles the incomplete, conflicting and uncertain data often met in building industry.

Now the needs are developments in the aggregation of different opinions from experts and in the assignment of the mass on data, according to their quality. We also have to distinguish precisely nominal functioning (service life assessment in hypergraph) and exceptional functioning (in FMEA).

Finally, the bounds of the studied system have to be defined in order to take into account the interactions in the whole building (it will be interesting to simulate the potential pathologies in a building, submitted to various weathering factors).

7 References

BOUCHON-MEUNIER, B. (1995) *La logique floue et ses applications*. Addison-Wesley, Amsterdam. pp.84-92.

British Standards Institution (1992). *Guide to Durability of buildings and building elements, products and components*. BSI, London. BS 7543.

DROMIGNY-BADIN, A. ROSSATO, S. YUE MIN ZHU (1997). *Fusion des données radioscopiques et ultrasonores, in Traitement du Signal*. vol.14, n°5: pp.499-510.

EPFL-LESO (1995) *Vieillissement des éléments de construction et coût d'entretien*. Office Fédéral des Questions Conjoncturelles: Berne.

EURIN, J. MARECHAL, J.C. et al. (1985) *Barriers to the prediction of service life of polymeric materials*, in Problems in Service Life Prediction of Building and Construction Materials (NATO ASI Series) Martinus Nijhoff Publishers.

FUNTOWICZ, S.O. RAVETZ, J.R. (1990) *Uncertainty and Quality in Science for policy*. Kluwer Academic Publishers: Dordrecht (The Netherlands)

GUMPERTZ, W.H. SCHWARTZ, T.A. (1996) *The Well-Designed Horse Carriage*, in Durability of Building Materials and Components 7 (Stockholm 1996), E & FN SPON, London. pp.1185-1194.

ISHIZUKA, Y. *The degradation and prediction of service life of building components*, in Durability of Building Materials, vol.1: pp.345-352.

KURTI, I. (1971) *Ubereinstimmung der Lebensdauer der Konstructionen und der Funktionellen Zeitgemässheit von Wohnhäusern*, in Periodica Polytechnica Architectura. vol.15 n° 3-4: pp.195-219.

LAIR, J. LE TENO, J.F., BOISSIER, D. *Prévision de la durabilité des systèmes constructifs du bâtiment*, in Fiabilité des matériaux et des structures (A. Mébarki, D. Boissier, D. Breysse) 2ème conférence nationale JN-FIAB'98. HERMES, 1998. pp.287-296

MARTIN, J.W. SAUNDERS, S.C. et al. (1996) *Methodology for predicting the service lives of coating systems*. Federation of Societies for coatings Technology. OPAC38 (1993) *Evaluation des besoins en entretien*. OPAC Isère. Publication interne.

PAPADAKIS, V.G. FARDIS, M.N. VAYENAS, C.G. (1992) *Effect of composition, environmental factors and cement-lime mortar coating on concrete carbonation*. in Materials and Structures vol.25 n° 149: pp.293-304.

PEARL, J. (1990) *Reasoning with Belief Functions : an Analysis of Compatibility*, in International Journal of Approximate Reasoning. n°4: pp.363-389.

PERRET, J. (1995), *Guide de la maintenance des Bâtiments*. Le Moniteur, Paris.

PIHLAJAVAARA, S.E. (1980) *Background and Principles of long term Performance of Building Materials*. Durability of Building Materials and Components (Ottawa). American Society for Testing and Materials, P.J. Sereda and G.G. Litvan, pp.5-16.

SARJA, A. VESIKARI, E. (1992) *Durability design of concrete structures*. RILEM TC130 CSL (Espoo).

SHAFER, G. (1976) *A Mathematical Theory of Evidence*, Princeton University Press, Princeton.

SIEMES, A. VROUWENVELDER, A. VAN DER BEUKEL A. (1985) *Stochastic modelling of building materials performance in durability*. in Problems in Service Life Prediction of Building and Construction Materials, (NATO ASI Series) Martinus Nijhoff Publishers.

122 MATERIAL INDUCED STRUCTURAL DETERIORATION AND MAINTENANCE STRATEGY

C.Q. Li

Monash University, Victoria, Australia

Abstract

Since material degradation is inevitable in reinforced concrete structures more attention should be drawn on structural performance assessment and the maintenance of the structure. This paper intends to provide an overview of problems of structural deterioration with focus on concrete structures. A technique to assess the life cycle performance of concrete structures is presented. Also presented is a basic idea of developing a reliability-based maintenance strategy. The developed strategy can be used to predict maintenance/repair time and ultimately the service life of concrete structures. The strategy will provide robust tools for rating and benchmarking of concrete structures, which would assist management in decision-making regarding their maintenance and rehabilitation. An example is given to illustrate how to assess the structural performance and to develop a maintenance strategy.

Keywords: Deterioration, concrete structures, maintenance, structural reliability

1 Introduction

The cost and consequences of structural failures are substantial, in particular, when people's lives are involved. The failure of concrete structures is usually attributed to the deterioration of structural strength. Structural deterioration stems from two basic sources: material degradation and the damage by external loads. Since both the material degradation and the external damage are inevitable in concrete structures, durability of concrete structures is an important issue in the design and operation of these structures. However, current design criteria for structural durability in most design codes, such as Australian AS 3600 (1994), are qualitative, which cannot

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determine the residual strength of a structure at a certain point of time in service and therefore cannot assess the structural performance over time. Due to structural deterioration, maintenance and repairs are necessary during the use of concrete structures. Otherwise the design service life of the structure cannot be fulfilled, which poses a risk to public life.

The importance of maintenance and repair of concrete structures is self evident. Just think about infrastructure around us. Concrete structures, such as buildings and bridges, constructed during 1960s and earlier, have deteriorated to an extent that many of them are unsafe for the increased service they are challenged to provide, such as the increased live load due to heavier equipment in a building or trucks on the road. The importance of maintenance can also be seen from the perspective of the cost on the maintenance and rehabilitation of infrastructure. In Australia it is currently estimated at about $30 billion per annum (6.7% of its GDP). Given this huge investment, together with an increasing scarcity of resources, it is essential that the funds be used as efficiently as possible.

To ensure that concrete structures function at a high level of quality and reliability, timely maintenance and rehabilitation of these structures are imperative. To have a rational and cost-effective maintenance strategy it is essential to be able to assess structural deterioration over time. Recently, control of the durability of concrete structures has gained increasing appreciation at the design and operation stages of these structures. Most of the current durability problems of concrete structures (for example, bridges, dams, and facades of buildings) could have been avoided if a systematic durability design and maintenance strategy had been developed. Such a design, however, requires both an overall methodology and detailed calculation models of actual deterioration processes. Unfortunately, both the methodology and deterioration models are not widely available and need to be developed.

This paper intends to provide an overview on problems of structural deterioration with focus on concrete structures. A technique to assess the life cycle performance of concrete structures is presented. Also presented is a basic idea of developing a reliability-based maintenance strategy. The developed strategy can be used to predict maintenance/repair time and ultimately the service life of concrete structures. The strategy will provide robust tools for rating and benchmarking of concrete structures, which would assist management in decision-making regarding its maintenance and rehabilitation. An example is given to illustrate how to assess the structural performance and to develop a maintenance strategy.

2 An overview of degradation problems

2.1 Material degradation

Literature shows that there are three major mechanisms that cause structural concrete deterioration (Warner, et al 1989): 1) abrasion, i.e., surface degradation of the concrete; 2) internal degradation of the concrete, such as Alkali Aggregate Reaction (AAR); 3) corrosion of the reinforcing steel in the concrete. Of these, steel corrosion is the most severe form of material deterioration with the capacity to threaten the safety of the structures. Since the problem of steel corrosion is dauntingly difficult to tackle, the term "concrete cancer" can be used to graphically refer to distress caused by steel corrosion.

Considerable research on the mechanisms of steel corrosion has been undertaken (Yeomans and Cook 1978; Sagoe-Crentsil 1989; Guirguis 1990). It has been understood that there are two basic mechanisms that trigger the corrosion of reinforcing steel in concrete. These are carbonation of concrete around the steel and the access of chloride ions to the surface of the steel. Steel corrosion is essentially a two-stage electrochemical oxidation process: the breakdown of a protective alkaline membrane occurs in an initiation stage, and the corrosion propagation proceeds in the second. It has also been known that, in addition to surrounding environments, the following factors affect steel corrosion, although the extent of effect varies: (1) water-cement ratio; (2) cement type; (3) concrete cover; and (4) concrete cracking.

Abrasion of the concrete surface involves initially the hardened cement paste on the surface of the structural member (Park and Pauley 1975). As the paste is worn away, firstly fine aggregates and then the coarse aggregates are exposed. Depending on the strength of the paste and subsequently on the bond and hardness of the aggregate, surface degradation may take place. It has been identified that the following factors affect the abrasion of concrete surface: (1) compressive strength; (2) aggregate properties; (3) finishing procedures; and (4) curing.

Alkali Aggregate Reaction (AAR) is a problem intrinsic to the concrete itself. Due to the constituents of concrete and environments around, some countries, such as the United Kingdom have more severe AAR problem than others, such as Australia (actually, in the UK and some other European countries, "concrete cancer" is referred to as AAR). Since 1940, many studies on AAR have been carried and are available in literature (e.g., Grattan-Bellew 1986). It is well known that conditions and factors that affect AAR are: (1) presence of reactive aggregates; (2) supply of sufficient alkalies; and (3) presence of adequate moisture.

It needs to be pointed out that research on material degradation is quite extensive and reliable information and experimental data have been produced and published. It is being recognised that a step forward from the research of material degradation to that of structural deterioration would lead to a cutting edge of the research in concrete durability (from the performance perspective of both materials and structures).

2.2 Structural deterioration

As has been noted, structural deterioration stems from two basic sources: material degradation and the damage by extreme hazards, such as in strong wind or an earthquake. This paper is only concerned with the material degradation. Structural deterioration could be pursued in a number of ways, according to performance requirements and limit states that are used in the design. For example, the deterioration in structural strength is when the strength of the structure decays to an unacceptable limit state. The deterioration in serviceability, such as deflection, is when the deflection exceeds an allowable limit state.

Structural deterioration is one of the least attended topic in structural reliability analysis (Melchers 1987; Rackwitz 1983). The most difficulty lies in lack of deterioration models. Existing models for structural deterioration are either too difficult to tackle mathematically or too simple to make any practical sense. From the viewpoint of concrete structures, steel corrosion in concrete usually represents the final stage of structural deterioration prior to collapse. However the majority of the research on steel corrosion is limited more to corrosion mechanisms than to residual strength. As a consequence, there are few stochastic models that could simulate the behaviour of deteriorating structures, a situation which has actually overlooked the random nature of the problem. A key step to establishing such a model is to identify critical factors that relate steel corrosion to its structural strength. But this is not easy in view of the fact that the deterioration rate depends largely on constituents of concrete, environments, structural geometry and loadings.

An ideal probabilistic model that takes into account both degradation and proven-strength could be the one as shown in Fig. 1 (Kameda and Koike 1975). As can be seen from Fig. 1 the degradation is represented by the leftwards shift of the probability density function of the structural strength. But the survival of the structure confirmed that its initial strength was higher than originally assumed (proven-strength). So the probability density function should be amended, conditional on its survival, as represented by the truncated tail in Fig. 1. It is obvious that sufficient experimental data are required to construct the probability density function of the structural strength as well as the accurate truncation of the function.

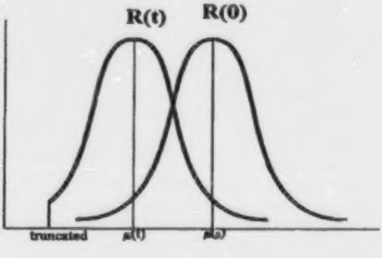

Fig. 1: Probabilistic model of deterioration (R = resistance)

3 Performance assessment

Since both material degradation and structural damage are inevitable, research on structural deterioration should be focused on how to assess structural performance over time and to predict the maintenance time for the structure and hence its service life (i.e., maintenance strategy, see below). Again, research in this regard is very limited and only recently progress has been made in Europe (Sarja and Vesikari 1996). In general, structural performance should be assessed using time-dependent structural reliability approaches since both the applied loads and the structural strength are not only uncertain but also time-variant. For this purpose, the basic theory of time-dependent reliability is briefly described below. Details of the theory will not be discussed in the paper but are available in textbooks, such as Melchers (1987).

3.1 Time-dependent reliability

A durability problem is a time-dependent reliability problem, in which all (or some) variables should be modelled as stochastic processes. For reliability problems involving stochastic processes, the structural reliability depends on the time that is expected to elapse before the first occurrence of the stochastic processes crossing out of the safe domain D, sometime during the service life $[0, t_L]$ of the structure. Equivalently, the probability of the first occurrence of such an excursion is the probability of structural failure $p_f(t)$ during that time period. Under some assumptions (see, Melchers 1987), the so-called "first passage probability" can be determined from

$$p_f(t) = p_f(0) + \int_0^t v\,d\tau \tag{1}$$

where $p_f(0)$ is the probability that the structure fails at time $t = 0$. For structural deterioration problem $p_f(0)$ is obviously very small and may be assumed to be zero, which means that it is unlikely that a structure deteriorates as soon as it comes into use. The main difficulty in application of equation (1) to practical problems is the determination of the outcrossing rate $v(t)$, to which only few solutions exist.

When the stochastic process $X(t)$ is a scalar process, the outcrossing problem becomes a down or up-crossing one and for structural deterioration it is obviously a downcrossing problem. So that the outcrossing rate in equation (1) can be replaced by a downcrossing rate, which can be evaluated directly by Rice formula (Rice 1954) as

$$v = v_a^- = \int_{\dot a}^{-\infty} (\dot x - \dot a) f_{x\dot x}(a, \dot x)\,d\dot x \tag{2}$$

where v_a^- is the downcrossing rate of the scalar process $X(t)$ relative to a, the threshold (limit state) to be crossed, $\dot a$ is the slope of a with respect to time, $\dot x(t)$ is the time derivative process of $X(t)$ and $f_{x\dot x}(x, \dot x)$ is the joint probability density function for X and $\dot X$. A solution to equation (2) can be expressed analytically, when the threshold a is not random (constant), as (Li and Melchers 1993a)

$$v = v_{a=const} = \frac{\sigma_{\dot x|x}}{\sigma_x} \phi\left(\frac{a-\mu_x}{\sigma_x}\right) \left\{ \phi\left(\frac{\dot a - \mu_{\dot x|x}}{\sigma_{\dot x|x}}\right) - \frac{\dot a - \mu_{\dot x|x}}{\sigma_{\dot x|x}} \Phi\left(\frac{\dot a - \mu_{\dot x|x}}{\sigma_{\dot x|x}}\right) \right\} \tag{3}$$

where $v_{a=const}$ stands for the downcrossing rate when the threshold is not random, $\phi(\ )$ and $\Phi(\ )$ are standard normal density and distribution functions respectively, μ and σ denote the mean and standard deviation of X and $\dot X$, represented by subscripts and "|" denotes the conditional probability. For a given stochastic process, all the variables in equation (3) can be determined as shown in Li and Melchers (1993a).

3.2 Practical application

In the context of structural reliability theory, assessment of structural deterioration is based on the probability of the violation of a durability limit state. At the time that the probability of deterioration failure is below an acceptable limit, maintenance is required or the service life of the structure is attained.

Let the performance of a reinforced concrete structure be represented by residual strength, e.g., the original flexural capacity, M_o, of a structural member, which is determined based on design codes. M_o is a nominal value of the flexural capacity (which is a random variable). Also let the time function φ(t) be used to describe structural deterioration. Thus the flexural capacity of the structure at time t, M(t), can be expressed as

$$M(t) = \varphi(t)M_o \tag{4}$$

where $\varphi(t) \le 1$ is known as deterioration function. For the structural member to be safe it should satisfy the following limit state of durability: $M(t) \ge M_a$, where M_a is a threshold of the flexural capacity beyond which the probability of structural failure due to structural deterioration is too high to be accepted. So whenever M(t) is less than the threshold M_a, the structure fails due to deterioration. This is a typical downcrossing problem and equations (1) and (3) are readily used to calculate the probability of this deterioration failure, with M(t) replacing X(t) and M_a replacing *a*,

$$v = \frac{\sigma_M}{\sigma_M}\phi\left(\frac{M_a - \mu_M}{\sigma_M}\right)\left\{\phi\left(-\frac{\mu_M}{\sigma_M}\right) + \frac{\mu_M}{\sigma_M}\Phi\left(-\frac{\mu_M}{\sigma_M}\right)\right\} \tag{5}$$

It may be noted that probability of structural failure due to material degradation has been separated from that due to external loads. This is because there is no evidence suggesting that material-induced structural deterioration is related to external loads, although damage-induced structural deterioration is associated with external load. The apparent assumption herein is that failures due to structural deterioration and applied loads are independent.

4 Maintenance strategy

There are basically two issues in developing a maintenance strategy. One is that when the maintenance is needed. The other is how to implement the maintenance, i.e., the repair techniques. This paper is only concerned with the former. To develop a maintenance strategy the key element is to determine the time when a structure needs maintenance. This is essentially a problem of making decision under uncertainty. The uncertainty comes from variations of material degradation itself, models used to assess the structural deterioration and so on. Because of this uncertainty and time-variant nature of the problem, only a time-dependent reliability approach is appropriate in developing the maintenance strategy.

There are basically two categories of maintenance policies: preventive policy and corrective policy.

4.1 Preventive policy

Under this policy, only routine inspection/repair is carried out, which is to prevent defects that may cause structural deterioration. So it is also called routine maintenance. Take a concrete building as an example. The routine maintenance includes patching concrete, repairing cracks, cleaning exposed parts and replacing small parts (non-structural). Guidelines for routine maintenance usually exist. The routine maintenance is usually carried out at a regular time interval which varies according to working conditions and environments. Since the time intervals for routine maintenance are more or less determined *a priori*, and since the cost associated with the maintenance is usually low, the preventive policy of maintenance is not discussed any further in the paper.

4.2 Corrective policy

Corrective policy is to retro-fit and/or restore the structural capacity due to material degradation or damage. Under this policy, only when a structure fails (defined in any way) would maintenance be carried out. The corrective maintenance involves considerable repair work. Compared with routine maintenance, corrective maintenance is less frequent and there are, in general, no guidelines to repair work. Until now the corrective maintenance strategy is, by and large, based on experience and local practice rather than on sound theoretical investigation.

Maintenance and repair based solely on experience may be more expensive and less safe than those based on a more rational material and structural assessment. Due to the substantial cost difference between preventive policy and corrective policy, it is very important to ensure that maintenance is carried out only when it is needed if a cost-effective maintenance strategy is to be achieved. This requires a cost-risk approach in developing a maintenance strategy.

A reliability-based maintenance strategy is to maintain a minimum acceptable reliability, β_m as shown in Fig. 2, throughout the lifetime of the structure. β_m can be determined by minimising the total life cycle cost of a structure to be built, including initial construction cost, maintenance cost and the cost of structural failure (Li 1995). In this strategy, the time, such as t_i in Fig. 2, for maintenance is determined in terms of a probability. That is only when a structure deteriorates below an acceptable reliability level, the maintenance intervention is necessary. So the key in corrective maintenance strategy is to determine the time the maintenance is required, i.e., t_i. In Fig. 2, β_T is the target reliability index. It represents the safety level built in the design codes.

The corrective maintenance strategy is consistent with the current practice or experience, which may be described as "If it ain't broke, don't fix it". "Broke" should be defined on a more sound scientific base as proposed herein, i.e., using a probabilistic limit state to define the failure (broke). It is the probability of the failure that is not acceptable rather than the actual failure itself, which is the conceptual difference to experience-based maintenance policies. When a structural fails it is usually too late to fix it.

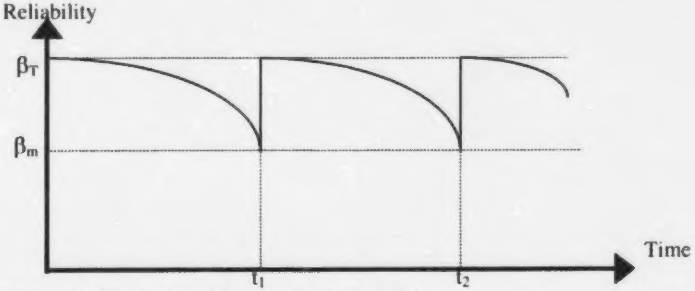

Fig. 2: Concept of life cycle performance (schematic)

4.3 Maintenance time

To determine the time at which maintenance is needed, an acceptable limit of risk should be determined beforehand. Obviously this risk can be represented by an acceptable probability of structural failure, $p_{fa} = \Phi(-\beta_m)$.

By assigning $t = t_M$ in equation (1), a unique relationship between the probability of deterioration failure, associated with a limit state requirement, and the maintenance time, t_M, is established (although an analytical solution is not available at moment). Therefore, for a given acceptable probability of deterioration failure p_{fa}, if $p_f(t) < p_{fa}$, no maintenance is necessary at time t. At the time, e.g., t_1, that $p_f(t_1)$ is greater than or equal to p_{fa} the maintenance time for the structure is determined, i.e., $t_M = t_1$. Alternatively if there is no maintenance, the design service life t_L of the structure is attained, i.e., $t_L = t_1$. When there are more than one time of maintenance, the design service life becomes the sum of time intervals between maintenance, i.e., $t_L = \sum_{i=1}^{n} \tau_i$, where n is the number of maintenance times that are required during the service life of the structure and τ_i is the time interval between two maintenance times, i.e., $\tau_i = t_i - t_{i-1}$. The probability of structural failure within the service life t_L is the union of the failures in each time interval. As has been seen the structural performance (durability) is closely associated with the service life of the structure. This is the nature of durability problem.

5 Worked example

A concrete building located at a beach is taken as an example to show how to assess structural deterioration and implement a maintenance strategy at the design stage. It is expected that the cantilever beam of the balcony of the building would suffer severe steel corrosion. In this case, the deterioration function of equation (4) is simply taken from Li and Melchers (1993b) which is of a form: $\varphi = 1 - A_r(t)$, where A_r is the area reduction rate of reinforcing steel over time. Data that are adopted in the calculation of failure probability are as follows: $\mu_M(t) = 30(1 - t^{0.5})$, $C_M(t_i, t_j) = (0.3\mu_M)^2 \, at \quad i = j$, M_a = 20 kNm. Using equation (5), the probability of structural failure due to corrosion induced deteriroation can be calculated, the results of which are shown in Fig. 3.

The property manager decides 3 times of maintenance will be carried out during the service life of the building. To determine each maintenance time, an acceptable probability of failure should be determined first. For a given minimum acceptable reliability index, $\beta_m = 2.33$, the acceptable failure probability is

$$p_{f_a} = \Phi(-\beta_m) = \Phi(-2.33) = 0.01 \qquad (6)$$

As can be seen from Fig. 3, at the time t_1 = 23.5 years, maintenance is required to keep the risk lower than accepted. After the maintenance, it was assumed the flexural capacity can be restored to 95% of the original capacity. Following the same procedure, the second and third maintenance times can be determined, which were t_2 = 20.3 (years) and t_3 = 17.6 (years). After the 3rd maintenance, the building would serve for another 14 year before the probability of deterioration is lower than accepted. If there is no correlation between each maintenance, i.e., the maintenance is to be carried out independently, the service life of the building is $t_L = 23.5 + 20.3 + 17.6 + 14 = 75.4$ years with a confidence of 96%. In other words, the probability of deterioration failure of the concrete building for a given service life of 75.4 years is 0.04.

As can be seen from the results, the service life of concrete structures is directly related to a structural performance (bending herein) and it is predicted in a probabilistic manner.

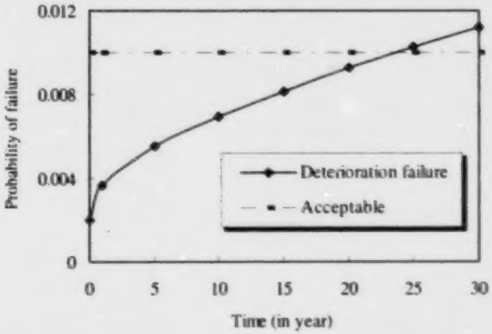

Fig. 3: Probability of deterioration failure

6 Conclusion

This paper has provided an overview on problems of concrete structural deterioration. A technique to assess the life cycle performance of concrete structures has been presented. Also presented is a basic idea of developing a reliability-based maintenance strategy. An example of a concrete building which is subject to steel corrosion has been taken to illustrate how to assess the structural performance and to develop a maintenance strategy. It can be concluded that a reliability-based maintenance strategy should be implemented at the design stage of concrete buildings.

7 References

Grattan, P.E. (Eds), (1986), Concrete Alkali-Aggregate Reactions, Proceedings of the 7th International Conference, Ottawa, Canada.

Guirguis, S., (1990), *Durable Concrete Structures*, TN57, Cement & Concrete Association of Australia.

Kameda, H. and Koike, T., (1975), "Reliability Theory of Deteriorating Structures", *J. Struct. Div., ASCE*, 101, (ST1), 295 - 310.

Li, C.Q., (1995), "Optimisation of Reliability-Based Structural Design", *Civil Engrg.Trans.*,37, (4), 303 - 308.

Li, C.Q. and Melchers, R.E., (1993a), "Outcrossings From Convex Polyhedrons for Nonstationary Gaussian Processes", *J. Engrg. Mech., A.S.C.E.*, 119, (11), 2354 – 2361.

Li, C.Q. and Melchers, R.E., (1993b), "Simulation of Corrosion of Reinforcing Steel in Concrete Structures", *Proc. 13th Aust. Conf. on Mech. of Struct. and Mater.*, 5–7 July, Wollongong, 517 – 524.

Melchers, R.E., (1987), *Structural Reliability Analysis and Prediction*, John Wiley and Sons, Chichester, U.K.

Park, R. and Pauley, T., (1975), *Reinforced Concrete Structures*, John Wiley & Sons, New York.

Rackwitz, R., (1983), "Reliability Formulation for Time-Variant Non-Deteriorating and Deteriorating Structural Systems", Proc. 4th Int. Conf. Applications of Statistics and Prob. in Soil and Struct. Engg., 609 - 619.

Rice, S.O., (1954), "Mathematical Analysis of Random Noise", *Bell. System Tech. J.*, 24, 46 - 156.

Sagoe-Crentsil K.K. *et al*, (1989), "Steel in Concrete: A Review", *Magazine of Concrete Research.*, 41, (149), 205 - 212.

Sarja, A and Vesikari, E., (1996), *Durability Design of Concrete Structures*, RILEM Report 14, E & FN Spon, pp165.

Standards Association of Australia, (1994), *Concrete Structures*, AS3600, Sydney.

Warner, R.F., Rangan, B.V. and Hall, A.S., (1989), *Reinforced Concrete*, Longman Cheshire, (3rd Edition), Melbourne.

Yeomans, S.R. and Cook, D.J., (1978), "Corrosion of Steel Reinforcement in Concrete Causes and Prevention", Proc. In. Conf. on Materials of Construction for Developing Countries, Bangkok, 443 – 451.

123 TOWARDS THE PRACTICAL EVALUATION OF SERVICE LIFE — ILLUSTRATIVE APPLICATION OF THE PROBABILISTIC APPROACH

K. Moser

Swiss Federal Laboratories for Materials Testing and Research,
Duebendorf, Switzerland

Abstract

Planning maintenance and repair of buildings requires detailed knowledge of the service lives of numerous building components. On the one hand, suppliers are often not in a position to give exact values for service life, let alone guarantee them, even under defined environmental and user conditions. Additionally, conditions over lifetime may differ from design assumptions and may change drastically over time. On the other hand, intricate methods and theories are often presented using rather elaborate methods for forecasting decay functions, distributions of service life and consequential costs.

This paper first discusses the conditions that govern the end of service life, considering the various but different properties of a simple building component. Thereafter, using probability density functions either from supplier's data, testing or estimates employing variations of the factorial method set out in ISO/CD 15686, simple examples are provided for a typical building component. Service life distributions for groups of similar building components are also established. These functions serve as a basis for forecasting financial requirements of maintenance activities. The proposed procedure does not rely on critical single value inputs, the results are easy to understand and contain much more information than just estimated mean values. Straightforward tools for processing these variables in the form of density functions are readily at hand.

Keywords: building component, density distribution, ISO/CD 15686, maintenance, service life, variables processing.

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1 Introduction

Planning maintenance and repair of buildings requires detailed knowledge of the service lives of the numerous building components. Suppliers are often not in a position to give exact values for service life, let alone guarantee them, even under defined environmental and user conditions. On top of this, conditions over lifetime may differ from design assumptions and may change drastically over time. New or modified products inherently can not offer results for medium or long-time use.

The results of service life calculations using average values only are easy to understand, but they do not give information on the effective distribution of service life. This clearly does not coincide with reality, as all elements of group will never fail at the same (average) point of time.

Intricate methods and theories often are presented using rather elaborate methods for forecasting decay functions, distributions of service life and consequential costs, e.g. Lounis (1998). These methods require large inputs of detailed data and special data processing programs. Their use today is only of significance for owners of a large numbers of structures, such as a state owning thousands of bridges or a real estate company managing hundreds of large buildings.

However, the use of probabilistic tools must not necessarily be restricted to such large populations of structures, but can be handled fairly easily with limited effort. In order to demonstrate this, simple examples are shown for a typical building component using probability density functions either from supplier's data, from testing or estimated employing variations of the factorial method set out in ISO/CD 15686-1 (1997).

These functions serve as the basis of the forecast of the financial demand for maintenance and renewal. The proposed procedure does not rely on critical single value inputs, the results are easy to understand and contain much more information than just estimated mean values. Straightforward tools for processing these variables in the form of density functions are readily at hand, e.g. VaP 1.6 (1996)

Another important point is the definition of the criteria, which govern the end of service life. These have first to be discussed before developing the probabilistic approach.

2 End of service life

2.1 Relevant properties

Service life in general is defined in ISO/CD 15686-2 (1998) as: "period of time after installation during which all conditions of a building or building part meet or exceed the performance requirements". It is worthwhile to take a closer look at the different groups of properties of a building part involved:

- Safety properties,
- Functional properties,
- Aesthetic (appearance) properties.

The first group of properties are the safety relevant or crucial properties, that are commonly defined using an appropriate safety factor. By law or by established professional principles this factor must not be impaired. The safety factor is defined by two values, i.e. resistance and load, that are combined into a relation. No visible signs normally exhibit possibly reduced resistance or increased load leading to an inferior safety factor. In some cases even new codes stipulating higher loads or reducing the resistance to be taken into account can lead to unsatisfactory limit states. Thus safety aspects must be complied with for building parts, but detrimental changes may not be obvious to the owner or user.

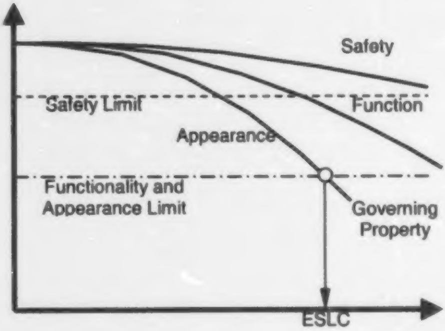


Fig 1: Schematic degradation of different groups of properties

A second group of properties are the functional properties required to fulfil the stipulated functions of the part, i.e. keep out wind, driving rain, coldness or to provide functioning wings for ventilation, glass panes to remain at least translucent, etc.

A third group of properties can be identified as aesthetic properties. Most of them deal with the superficial appearance of the part, be it a painted or of natural surface, a windowpane staying fully transparent or turning cloudy, etc. This group of properties typically sets off at very high levels for new parts, but naturally decreases towards much lower accepted levels during service life.

Taking the definition of ISO/CD 15686-2 (1998) literally, would mean that the end of service life is defined by any of these very different properties falling below the level specified, see Figure 1. This may be very sensible for safety conditions but, in the author's opinion, not necessarily for functional and aesthetic properties.

2.2 Combination of functional and aesthetical properties

Functional and aesthetical properties may be defined by a number of parameters. Instead of handling each parameter as critical on its own, which would make it dominant for service life, a combined value should rather be considered. It is proposed, that a simple average may be used (say 50% of "new") possibly combined with individual minimum values, which should be set relatively low (say 30% of "new"). Such an approach results in a judgement similar to the one performed by the user: One characteristic property may degrade to a relatively low level, without rejection of the part as a whole. If several properties degrade simultaneously, rejection occurs at a much higher level.

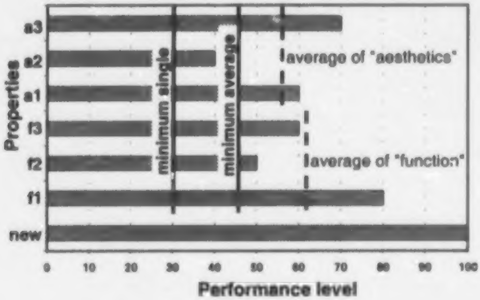


Fig. 2: Discussion of limits for different properties

3 Simple deterministic approaches

One simple approach to service life consists of giving just one plane figure for the estimated service life for the respective component.

Somewhat more advanced is the approach according to ISO/CD 15686-1 (1997): Average values for the reference service lives *RSLC* of the different building parts are set up and combined with factors according to the following equation:

$$\text{ESLC} = \text{RSLC} \times A \times B \times C \times D1 \times D2 \times E \times F \qquad (1)$$

From this equation one single value for the estimated service life of the component *ESLC* is derived. (The short definitions of the other variables can be found in Table 1.) It is quite evident, that the real behaviour of a group of similar building parts can not be characterised significantly by one single value. The service lives of the parts are not at all identical but exhibit a more or less large scatter around an average value.

These two and similar simple approaches can be helpful for a first orientation and may be satisfying for rough estimates. For more sophisticated planning, as for the proper scheduling of larger maintenance funds, they are clearly unsatisfactory.

4 Probabilistic approach

Hence a more detailed analysis is generally required. In most cases, this is a probabilistic approach. Figure 3 shows a schematic density curve based upon experience or manufacturer's data. However, if sufficient data from experience, from testing or from the manufacturer is not available, a density curve of the estimated testing or from the manufacturer is not available, a density curve of the estimated service life can be defined by professional estimate or by the (recursive) Delphi method. This is best done by setting values to the average (50%) and to the minimum and maximum values (typically chosen as fractiles of 5% and 95% of all elements). The fractiles are partial integrals of the density function, whose total integral is 1.0 or 100% equal to all elements included.

The recursive Delphi method uses first inputs by experts. After due consideration of the results from comparisons and processing of the values of the first input, the experts are invited to give a modified second and usually much more precise input. This procedure has proved to give extraordinary good results in fields with lack of sufficient statistical data (e.g. in risk engineering).

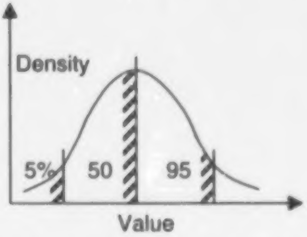


Fig. 3: Visualised typical fractiles defining density distributions

5 Numerical example

In order to demonstrate the procedure, a simple numerical estimate is calculated based on the example offered in ISO/CD 15686-1 (1997) for softwood windows.

5.1 Estimated service lives for the windows in all four faces

The basis of the numerical example is a squared building of a length of 50 m, a width of 25 m and a height of 30 m. The long sides are facing due south and north respectively. The windows of the four façades of the building are treated separately. Table 1 shows the arbitrarily assumed relevant conditions for all factors and faces. Therefrom the factors for the three fractiles 5%, 50% and 95% are defined in the sense of the Delphi method, in this case based on the factors and their description given in ISO/CD 15686-1 (1997).

Table 1: Fractile values of factors to calculate design life

Factor	Face	Relevant conditions	Factors for the fractiles 5% / 50% / 95%
A Quality of component	all	general variations of components	1.2 / 1.5 / 1.8
B Design level	all	good, identical value	1.2
C Work execution level	all	general variation, but insufficient quality repaired	1.0 / 1.2 / 1.5
D1 Indoor environment	S	occasional risk of condensation	0.9 / 1.0 / 1.2
	W	medium risk of condensation	0.8 / 0.9 / 1.1
	N	high risk of condensation	0.7 / 0.8 / 0.95
	E	medium risk of condensation	0.8 / 0.9 / 1.1
D2 Outdoor environment	S	occasional cycling dry / damp	0.8 / 1.0 / 1.3
	W	regular cycling dry / damp	0.6 / 0.8 / 1.0
	N	sheltered from rain	1.0 / 1.2 / 1.5
	E	occasional cycling dry / damp	0.8 / 1.0 / 1.3
E In use conditions	S	occasional access by children ^{1})	0.8 / 1.0 / 1.2
	W	regular access by children ^{1})	0.6 / 0.8 / 1.0
	N	occ. / reg. access by children ^{1})	0.7 / 0.9 / 1.1
	E	occasional access by children ^{1})	0.8 / 1.0 / 1.2
F Maintenance level	all	painted on judgement of caretaker	0.9 / 1.0 / 1.1

Note 1) according to example in ISO/CD 15868-1 (1997)
(other descriptions for wear and tear may appear more realistic).

The factors for *C* reflect the fact that a quality of workmanship usually is such that all parts are not likely to be repaired to a degree sufficiently adequate to meet the requirement, whereas those exceeding the requirement are naturally left at their higher level. This procedure leads typically to asymmetric distributions (Figure 4).

The values for the fractiles given in Table 1 are approximated by the functions given in Table 2 for ease of processing. The functions chosen represent those generally used: determistic-, normal-, lognormal- and Gumbel- (extreme-value-) distributions. The program used for the data processing, VaP 1.6 (1996), supports 11 types of distributions plus the possibility of user defined functions. The variables *m* and *s* are the first and second moments of the respective distributions (see VaP 1.6 (1996) for additional details and system requirements).

Table 2: Functions for approximating the estimated fractile values

Factor	**Type of Distribution**	**Face** **South** *m / s*	 **West** *m / s*	 **North** *m / s*	 **East** *m / s*
RSLC	Deterministic	25 years	25 years	25 years	25 years
A	Normal	1.5 / 0.185	1.5 / 0.185	1.5 / 0.185	1.5 / 0.185
B	Deterministic	1.20	1.20	1.20	1.20
C	Gumbel	1.25 / 0.10	1.25 / 0.10	1.25 / 0.10	1.25 / 0.10
D1	Lognormal	1.05 / 0.10	0.95 / 0.10	0.80 / 0.10	0.95 / 0.10
D2	Lognormal	1.05 / 0.20	0.80 / 0.20	1.25 / 0.20	1.05 / 0.20
E	Normal	1.0 / 0.12	0.80 / 0.12	0.90 / 0.12	1.0 / 0.12
F	Normal	1.0 / 0.06	1.0 / 0.06	1.0 / 0.06	1.0 / 0.06
ESLC (years)	Lognormal [1])	62.0 / 20.4	34.2 / 11.8	50.6 / 14.8	56.1 / 18.6

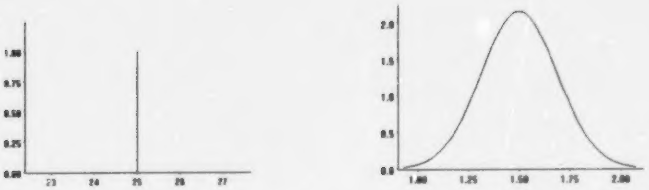
Note: [1]) close fit

The functions for the south windows are plotted in Figure 4 as representative illustration for all four faces. The average of *ESLC* (first moment *m*) can be found by simply multiplying the *RSLC* by all *m*-values. The second moment *s* can e.g. be determined using the appropriate equations from statistics or by carrying out Monte Carlo simulations. The results were calculated by both methods using VaP 1.6 (1996). The results of the direct method are shown on the last line of Table 2. The average results of two runs of a Monte Carlo simulation match these results by a maximum difference on the average value of 0.1 years. These simulations yielded detailed results as shown graphically in Figure 5. The absolute densities are derived by dividing the relative densities shown on the vertical axis by the number of runs, in this case by 100, 000.

5.2 Comparison of four facades

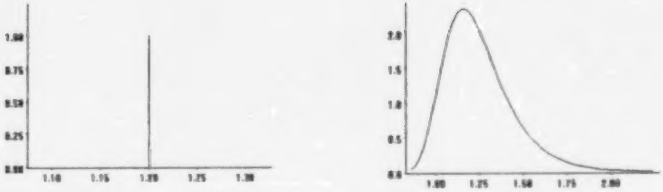
The results for the estimated service lives of the four facades as provided in Figure 5 are different in several aspects. First, one notices the different widths of the distributions, all plotted against the same horizontal scale. The spread is largest for the south face and narrowest for the west face. Some of this effect is relative: Due to the high average value, the same relative spread is larger in absolute terms.

The west face shows as expected the shortest service life, mainly due to the unfavourable outdoor climate and the in-use conditions. The effect of the higher risk of condensation, assumed for the north face indoor climate, is offset by the more favourable outdoor climate. The main difference originates from the in-use conditions. Both effects combined yield some 15% less estimated service life.



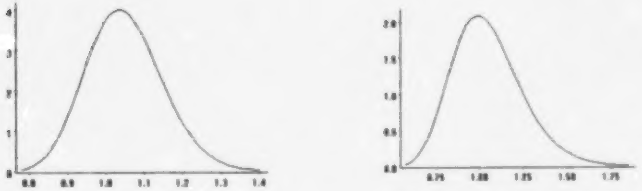
ESLC: Estimated Service Life

A: Quality of component



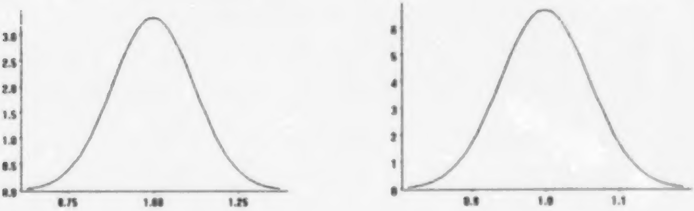
B: Design Level

C: Work execution level



D1: Indoor environment

D2: Outdoor environment



E: In use conditions

F: Maintenance level

Fig. 4: South façade: variables defined as functions

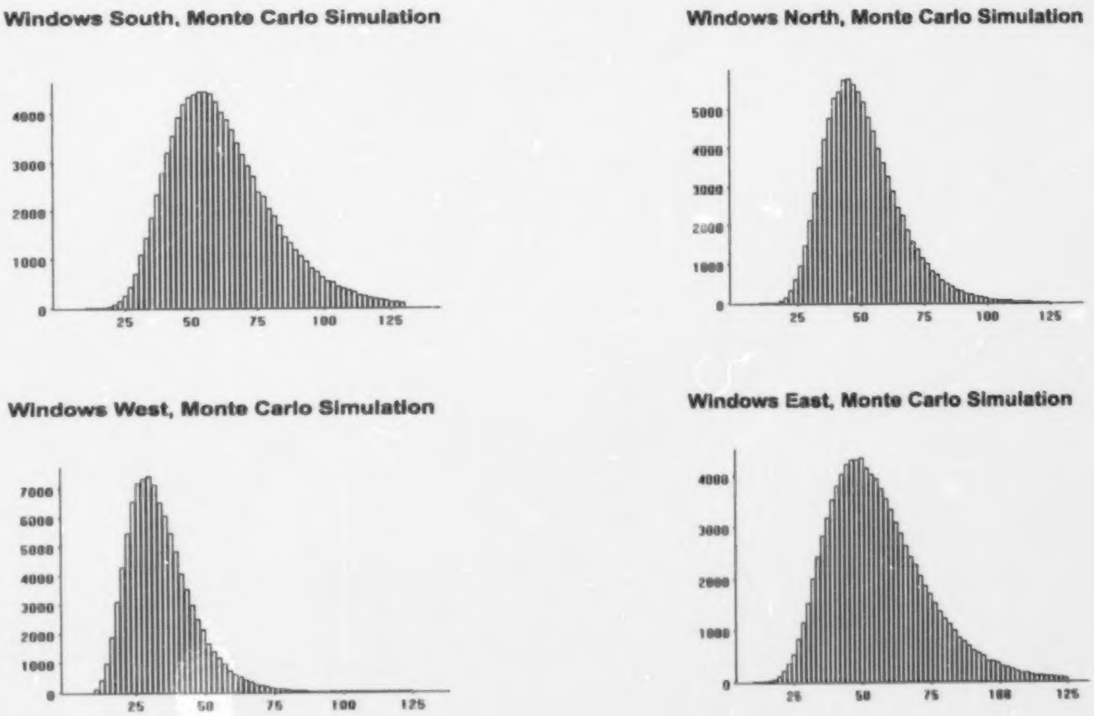


Fig. 5: Density distribution for the windows in the four faces

5.3 Financial demand

For the estimation of maintenance funds, the functions for the service lives of similar building components have to be superimposed. In general, this has to be done for all parts of a building considered. For the superposition, costs have to be allocated to the different groups of building components. In this example, typical results are shown in which only the superposition of all window areas to be replaced is considered in the calulations. It is assumed for this purpose, that the windows cover 40% of the area of the respective facades.

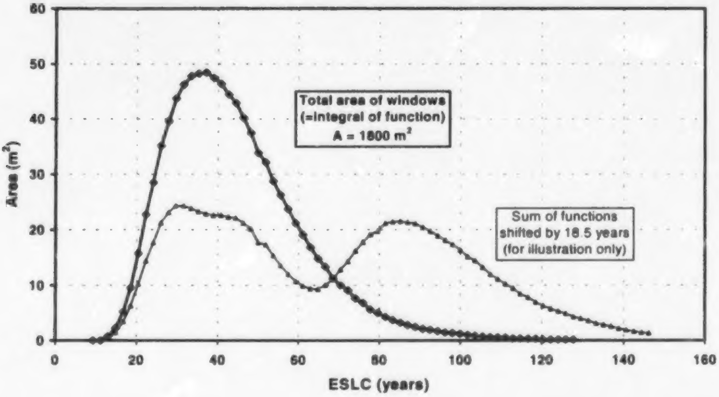


Fig. 6: Financial demand (shown as window area to be replaced)

The superposition yields an asymmetric function having a steep increase up to a peak demand of replacements after 37 years of 48 m^2/year. Then the demand decreases at a gentle slope down to 10 m^2/year after 70 years. In a next step, the same service life functions have to be applied to the replaced windows, and the results of the multiple replacements are summed up, leading to a fairly constant replacement function. This step is omitted here for clarity.

The lower curve depicts another sum function based on the same basic data, but the subfunctions of the individual façades being shifted apart by 18.5 years each for illustration purposes, thus resulting in a double hump shape. This curve is fairly representative for the superposition of elements with a remarkably different service life.

In general, the financial demand for similar components tends to merge into a one-peak function, whereas the superposition of the functions of all different parts of a building is more likely to result in several peaks or even a relatively steady demand over the lifetime of the building considered, starting at a certain age of the building.

6 Conclusion

The use of probabilistic tools for planning maintenance or replacement costs is no longer only restricted to projects having large funding requirements or numerous assets. Making full use of the information, e.g. given in ISO/CD 15686 and modified by professional opinion, permits the use of variables instead of deterministic factors in the equation for the estimated service life. The results give a much more detailed insight into the service life of building components involved and allow a far better planning of the investments required.

7 References

ISO/CD 15686-1 (1997) *Buildings: Service Life Planning, Part 1 – General Principles*, ISO/TC59/SC3, document N391, 60 p.

ISO/CD 15686-2 (1998) *Buildings: Service Life Planning, Part 2 – Service Life Prediction Principles,* ISO/TC59/SC14, document N3, 23 p.

Lounis, Z., Lacasse M.A., Siemes A.J.M., Moser K. (1998*) Further steps towards a quantitative approach to durability design,* CIB World Building Congress, Gävle, Sweden, pp. A280-A289.

Schneider, J. (1993) *Sicherheit und Zuverlässigkeit im Bauwesen* (Safety and Reliability in Building Industry, includes large (English) bibliography), vdf, Zürich and B.G. Teubner, Stuttgart, 182 p.

VaP 1.6 for Windows™ (1996) *Short course in Variables Processing*, ETH (Swiss Federal School of Engineering) Zürich, Manual 13 p. in English; Papers 33 p. and Examples 22 p. in German.

124 EVALUATING THE RELIABILITY OF STRUCTURAL MASONRY ELEMENTS USING THE RESPONSE SURFACE TECHNIQUE

L. Schueremans,[1] D. Van Gemert,[1] and M. Maes[2]

[1]Catholic University of Leuven, Belgium; [2]University of Calgary, AB, Canada

Abstract

In historical buildings, the load bearing capacity is often doubtful due to deterioration processes or external actions. To decide whether or not these structural elements need to be strengthened, a probabilistic method is searched for. Indeed, as none of the parameters is known exactly - material characteristics, imposed loads due to the new or altered destination of the building, uniformity of material properties, ...- a probabilistic method is developed. The goal of this paper is to illustrate the applicability of probabilistic methods for the safety assessment of historical masonry constructions. This objective is illustrated on a masonry shear panel under horizontal loading. A 2D non-linear finite element model is used for the numerical analysis. The outcome of the numerical experiments provide the input for the response surface method. The surface is fit into the limit state function, that allows the probability of failure to be calculated using a First-Order Reliability Method and the Monte Carlo method.

Keywords: failure modes, FORM, homogenisation, load-bearing capacity, masonry, probability of failure, reliability analysis, Response Surface technique.

1 Introduction

Extremely idealised structures can be studied in an analytical way, in deterministic as well as in probabilistic context. A real structure however has a high degree of mechanical complexity. Therefore, even deterministic analyses can be conducted only by numerical algorithms (finite element analyses) (Giannini and al. 1996). Besides the mechanical complexity, the analysis of historical masonry constructions is mainly characterised by the uncertainties related to the material parameters and geometry. Among the possible approaches to this type of problem are the stochastic finite element method and Monte Carlo simulation. The latter, although very general in principle, is limited in practice by the heavy computational demand necessary to build up statistically significant samples. The Response Surface method,

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extensively used in statistics, and proposed a few years ago for structural reliability problems (Faravelli 1989; Veneziano and al. 1989), represents under certain conditions a valid alternative for simulation. This method not only provides information on the randomness of the response, but also quantifies the contributions to the response uncertainty from different sources of uncertainty. Therefore, in combination with a finite element calculation and a reliability analysis, this method can be characterised as a stochastic finite element method to assess both the response uncertainty and its dependence on the randomness of the design variables.

The method searched for has to be able to solve problems of realistic dimensions. Therefore, a combination of two techniques is presented. To restrict the amount of elements in the finite element analysis, homogenisation has already proven its reliability in a deterministic sense. The method is extended introducing the spatial variability of the masonry material properties and geometry. In the Response Surface method, the main variables are reduced to a limited set, covering the major uncertainties. The effect of all other parameters, grouped in an appropriate subset, is accounted for with a statistical treatment. A well-organised amount of suitable numerical experiments provides the outcome required for estimating the response surface.

The evaluation of the global probability of failure of a rectangular shear panel requires a mechanical model capable of describing the complex behaviour of the structure under increasing external forces up to its failure. A 2D non-linear finite element model allowing for local crushing and fracturing of the masonry has been adopted for the numerical analysis.

Once the response surface is established, it is fit into the limit state function to calculate the probability of failure of the shear panel under horizontal loading. For that purpose, a FORM analysis, combined with an Importance Sampling Monte Carlo algorithm is used.

2 Response surface technique

2.1 Governing relations

A reliability analysis requires the definition of the limit state function, depending on the main random variables **X** which control the structural behaviour. The chosen limit state (LS) function is of the form:

$$\text{LS: } g(\mathbf{X}) = Y(\mathbf{X}) - F_H \qquad (1)$$

$Y(\mathbf{X})$ is the horizontal load which causes the shear panel failure, and the random variable F_H is the actual horizontal load on the structure. Using a FORM analysis and a Monte Carlo simulation, the probability of failure is calculated, with respect to the limit state function as expressed in eq. 1 (Melchers 1987). The variable **X** represents the random fields of material parameters that, after the finite element discretization, become random variables representing the material parameters at the Gauss points.

For complex structures with realistic dimensions, the problem is that the response is not known beforehand and has to be estimated from data and/or numerical experiments. Then we have now the problem that:

$$Y = Y(\mathbf{X}) \tag{2}$$

A major disadvantage of the Response Surface method lies in the exponential growth ($\sim 2^n$), with respect to the number (n) of terms in **X**, of the number of experiments required to determine the parameters of the surface. In order to overcome this limitation, **X** is subdivided into two sub-vectors: $\mathbf{X}=[\mathbf{X}_1, \mathbf{X}_2]$ (Breitung and Faravelli 1996; Giannini and al. 1996). In order to separate the **X** vector into two sub-vectors, the random fields of the material parameters are expressed in the form:

$$X_i(\xi,\eta) = v_i.w_i(\xi,\eta) \tag{3}$$

Where i is the number of the material parameter and (ξ,η) the co-ordinates. V_i is a random variable expressing the spatial mean of $X_i(\xi,\eta)$ on the whole structure, whereas $w_i(\xi,\eta)$ indicates a random field with unit mean, which accounts for the spatial variability of $X_i(\xi,\eta)$. Only the random variables v_i are included in the vector $\mathbf{X}_1$ of the parameters which control the Response Surface. In this example, $\mathbf{X}_1$ consists of 4 random variables representing the major material properties of the masonry: f_c (compressive strength), f_t (tensile strength), E (initial modulus of elasticity) and E_s (softening modulus after failure).

The spatial variability of these material properties – the random field description – and the description of the localised defects are included in the vector $\mathbf{X}_2$. This option is taken, mainly to reduce the computational effort. It may be justified by the fact that if more data become available, they will more likely concern the entire wall more than the spatial variability of the material properties or the distribution of the local defects (Giannini and al. 1996). Then, the outcome of the experiments can be written as:

$$Y(\mathbf{X}) = Y_1(\mathbf{X}_1) + \varepsilon_0(\mathbf{X}_2) \tag{4}$$

in which $\varepsilon_0(\mathbf{X}_2)$ is an error term represented by a zero-mean Gaussian random variable, accounting for the exclusion of $\mathbf{X}_2$ in $Y_1(\mathbf{X}_1)$. This intrinsic randomness is the so-called "pure error".

The Response Surface technique substitutes the actual and unknown function $Y_1(\mathbf{X}_1)$ with an approximating polynomial of low order whose coefficients must be determined from the results of suitable experiments. In this case the experiments are carried out using a non-linear finite element analysis of the shear panels to determine the failure load corresponding to specific samples of $\mathbf{X}_1$.

In this example a polynomial of second order is chosen, resulting in the following formulation:

$$Y = \left\{\theta_0 + \mathbf{X}_1^T\theta_1 + \mathbf{X}_1^T\theta_2\mathbf{X}_1 + \varepsilon_L(\mathbf{X}_1)\right\} + \varepsilon_0(\mathbf{X}_2) \tag{5}$$

in which θ_0, ϑ_1 and θ_2 are the regression coefficients and ε_L is the "lack of fit" error term, accounting for the 2^{nd} order approximation of the Response Surface. For ease of formulation and calculation, the 2nd order polynomial or Response Surface formulation is put into matrix notation:

$$\mathbf{Y} = \mathbf{A}\theta + \varepsilon_L + \varepsilon_0 \tag{6}$$

where $\boldsymbol{\theta}$ is the vector of the model parameters θ_k (k= 1…1+n+n(n+1)/2) and **A** is the observation matrix that contains constant, linear, quadratic and cross-combinations of X_i (i = 1,…,n_e and n_e is the number of experiments). Fully outlined, Eq. 6 in general becomes:

$$\mathbf{Y} = \begin{bmatrix} 1 & x_1^{(1)} \cdots x_n^{(1)} & x_1^{(1)} x_2^{(1)} \cdots x_{n-1}^{(1)} x_n^{(1)} & \left(x_1^{(1)}\right)^2 \cdots \left(x_n^{(1)}\right)^2 \\ 1 & x_1^{(2)} \cdots x_n^{(2)} & x_1^{(2)} x_2^{(2)} \cdots x_{n-1}^{(2)} x_n^{(2)} & \left(x_1^{(2)}\right)^2 \cdots \left(x_n^{(2)}\right)^2 \\ \vdots & \vdots & \vdots & \vdots \\ 1 & x_1^{(n_e)} \cdots x_n^{(n_e)} & x_1^{(n_e)} x_2^{(n_e)} \cdots x_{n-1}^{(n_e)} x_n^{(n_e)} & \left(x_1^{(n_e)}\right)^2 \cdots \left(x_n^{(n_e)}\right)^2 \end{bmatrix} \begin{bmatrix} \theta_0 \\ \theta_1 \\ \theta_2 \\ \vdots \\ \theta_{\left(1+n+\frac{n(n+1)}{2}\right)} \end{bmatrix} + \begin{bmatrix} \varepsilon_L^{(1)} \\ \varepsilon_L^{(2)} \\ \varepsilon_L^{(3)} \\ \vdots \\ \varepsilon_L^{(n_e)} \end{bmatrix} + \begin{bmatrix} \varepsilon_0^{(1)} \\ \varepsilon_0^{(2)} \\ \varepsilon_0^{(3)} \\ \vdots \\ \varepsilon_0^{(n_e)} \end{bmatrix} \tag{7}$$

where **Y** is the vector of the actual outcomes or observed responses (e.g.: obtained by the finite element analysis). It will be assumed that the random errors have zero mean, are non-correlated and have variances: $\sigma_{\varepsilon_L}^2, \sigma_{\varepsilon_0}^2$. Given n_e experiments the solution of eq. 7 provides the least square estimates of the model coefficients θ_k (Petersen 1985). The estimated vector of the regression coefficients is then

$$\theta = \left(\mathbf{A}^T \mathbf{A}\right)^{-1} \mathbf{A}^T \mathbf{Y} \tag{8}$$

The estimated value of the response vector **Y** results in

$$\hat{\mathbf{Y}} = \mathbf{A}\left(\mathbf{A}^T \mathbf{A}\right)^{-1} \mathbf{A}^T \mathbf{Y} \tag{9}$$

2.2 Design of experiments

For first-order approximations the design variables must be selected to satisfy orthogonality property

$$\left(\mathbf{A}^T \mathbf{A}\right) = c\mathbf{I}\,, \tag{10}$$

with: **I** = identity matrix and c = any constant. In fact, this minimizes the variance of the estimates θ_0 and θ_1 (Petersen 1985). The most often used orthogonal designs are the 2^n factorial designs [i.e. for two design variables, n=2: $\xi_{s1}^T = (-1,-1), \xi_{s2}^T = (-1,1), \xi_{s3}^T = (1,-1), \xi_{s4}^T = (1,1)$] and the fractional factorials.

An efficient class of design for fitting second-order response surface is the class of "central composite terms", leading to the central composite design (Montgomery 1997). This design consists of a 2^n factorial design, with each factor at the two-level -1 and +1, augmented by 2n points placed at the coordinates -α and +α of the axes (axial points). The axial points also called "star points" allow the design to fit higher order surfaces without the computational effort associated with a 3^n factorial design (Yoa and Wen 1996).

The estimation of pure error (Eq. 6) would require the replication of the 2^n factorials. Alternatively "augmented 2^n factorials" are obtained by adding n_2 points at the center of the design. Therefore, the design contains $2^n + 2n + n_2 = n_e$ points. The values of α and n_2 are chosen by the experimenter to achieve the design properties (Petersen 1985). In the example of the shear panel, the following amount of experiments is chosen: $2^n + 2n + n_2 = 2^4 + 2x4 + 8 = 32$. The distance α of the axial points from the design center is provided by means of the rotatability requirement (Petersen 1985; Montgomery 1997). For n equal to four, α equals:

$$\alpha = \left(2^n\right)^{1/4} = 2 \tag{11}$$

For the optimal design of experiments, use is made of the Matlab built-in m-files (Braley 1995). Two iterative algorithms are available to create optimal designs of experiments. These designs maximize the determinant of Fisher's information matrix **A'A**.

2.3 Testing the lack of fit

Depending on the choice of experiments, the obtained response surface differs. To be able to check the lack of fit of the response surface, an F-test is performed. For a detailed description the reader is referred to (Montgomery 1997; and Yoa and Wen 1996; Breitung and Faravelli 1996). If the model passes this test, it might be adopted. To select the best model among those which are not rejected by the test, the use of the statistic λ is proposed (Faravelli 1989), defined as:

$$\lambda = \sqrt{\frac{\sigma_{\varepsilon_L}^2 \nu_0}{\nu_L \sigma_{\varepsilon_0}^2}}, \ where: \begin{cases} \nu_L = n_e - 2n - n_2 - 1 \\ \nu_0 = n_2 - 1 \end{cases} \tag{12}$$

If there is no lack of fit, the expected value of λ is given by (Montgomery 1997):

$$E[\lambda] = \sqrt{\frac{\nu_0}{\nu_0 - 2}} = \sqrt{\frac{n_2 - 1}{n_2 - 3}} \approx 1 \tag{13}$$

For the last approximation we assume that n_2 is not too small. The models, whose λ-values are near unity, are therefore taken as the best fit.

3 Homogenisation technique – Stochastic extension

In stead of making pre-assumptions about the correlated random field description given to the elastic properties (E, E_S) of the masonry shear wall similar to (Giannini and al. 1996), they are derived from the natural spatial correlation structure of a homogenised basic cell (Huyse and al.). Therefore, a stochastic extension of the homogenisation technique described in (Lourenço 1996) is made.

The structural analysis of historic masonry is affected by substantial uncertainties. The main uncertainties are: physical uncertainty associated with the strength and stiffness of bricks and mortar joints, and geometric uncertainty regarding the thickness of the head and bed joints, i.e. vertical and horizontal mortar joints respectively. Starting from bricks and joints a homogeneous basic cell is constructed. The two dimensional translation of this process is summarized below. For a full description, the reader is referred to (Huyse and al.; Lourenço 1996; Schueremans 1998). The objective is to obtain a macro or homogenised constitutive relation between homogenised stresses $\boldsymbol{\sigma}$ and homogenised strains $\boldsymbol{\varepsilon}$:

$$\boldsymbol{\sigma} = \mathbf{C}^h \boldsymbol{\varepsilon}, \quad \textit{where} \quad \boldsymbol{\sigma} = \lfloor \sigma_x, \sigma_y, \sigma_{xy} \rfloor \; \textit{and} \; \boldsymbol{\varepsilon} = \lfloor \varepsilon_x, \varepsilon_y, \varepsilon_{xy} \rfloor \tag{14}$$

Here $\mathbf{C}^h$ is the homogenised stiffness matrix, which will be obtained from the micro-constitutive relations, the constitutive relations of the bricks and mortar joints. Therefore, a homogenisation process is performed in two steps: first a homogenisation is performed along the x-axis. The resulting material becomes a layered composite (figure 1). Afterwards, the same procedure is repeated along the y-axis. Details of the homogenisation process can be found in (Lourenço 1996). The equation for the linear elastic stiffness matrix of the equivalent continuum, homogenised along one axis, reads:

$$\mathbf{C}^h = \left[\sum_i p_i \left(\mathbf{P}_t - \mathbf{C}_i \mathbf{P}_e \right)^{-1} \right]^{-1} \sum_i p_i \left(\mathbf{P}_t - \mathbf{C}_i \mathbf{P}_e \right)^{-1} \mathbf{C}_i \tag{15}$$

where p_i is the ratio of the thickness of layer *i* over the length of the basic cell, $\mathbf{C}_i$ is the stiffness matrix of layer *i* and $\mathbf{P}_t$ and $\mathbf{P}_e$ are projection matrices, table 1.

The homogenisation technique has already proven its reliability in the deterministic sense. The method is extended introducing the variability of geometry and physical properties.

Figure 1 shows the two step homogenisation process (Lourenço 1996). Because of simplifications in Lourenço's model, slightly different results are obtained for the elastic stiffness coefficients [C], depending on the homogenisation sequence: homogenisation along x-axis first, followed by homogenisation along y-axis, or vice-versa.

Table 1: Projection matrices P_t and P_e for homogenisation along x- and y-axis

Projection matrix	Homogenisation along x-axis	Homogenisation along y-axis
P_t	Diag[0 1 0]	diag[1 0 0]
P_e	Diag[1 0 1]	diag[0 1 1]

However, the difference is small for a wide range of values for the material properties.

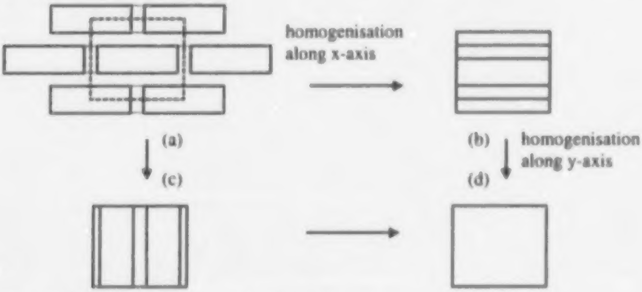

Fig. 1: Two step homogenisation process

The bricks as well as the mortar joints are assumed isotropic. Every basic cell consists of 5 different bricks, 4 of them only partly present in this basic cell. The stiffness of every brick (E_b) is considered random, introducing 5 normally distributed, non-correlated random variables. The mortar joints are divided into 6 different joints, splitting up head and bed joints. The variability of the joints is dual: the physical properties of the mortar are considered random (E_m) , introducing 6 non-correlated lognormal stochastic variables, and the geometry - governed by the thickness (t_m) - is taken random as well. The geometry introduces another 6 non-correlated lognormal stochastic variables. As the scatter in the geometry of the prefabricated units is believed to be small, the geometrical randomness will be governed by the randomness in the thickness of the mortar joints. They represent the workmanship of the masonry. The different variables are outlined in figure 2. The random variables are summarised in table 2.

The stiffness matrix [C] is calculated for the basic set of variables as outlined in table 2. As the homogenised material is no longer isotropic, the full 2D matrix of elastic moduli is given. The result for the xy-homogenisation - results of the yx-homogenisation are similar - is given below, based on 40,000 simulations:

$$\mu_{C,xy} = \begin{bmatrix} 12297 & 948 & 0 \\ & 8102 & 0 \\ symm & & 3054 \end{bmatrix} \quad and \quad Cov(\mathbf{C}_{xy}) = \begin{bmatrix} 5.89 & 11.51 & 0 \\ & 11.54 & 0 \\ symm & & 10.27 \end{bmatrix} \qquad (16)$$

As already mentioned, for neighbouring basic cells, some of the bricks or mortar joints will overlap. This introduces a natural spatial correlation between the stiffness moduli of different basic cells. When the correlation can be established for the neighbouring basic cells, it can easily be extrapolated to basic cells on a bigger distance from each other. Basically, 3 spatial correlations need to be calculated, for the closest neighbours in the x, y and diagonal position, see arrows figure 3. The correlations are calculated for the basic set of values as mentioned in table 1, using 10.000 simulations. For the simulation of the correlated stochastic field of the material parameters later on, only the diagonal values of the correlation matrices are used. This is a so-called first order analysis, introduced for reasons of computational effort (Huyse and al.).

Table 2: Basic set of variables

Random Variables	Pdf	Mean μ	cov [%]
$E_{b1},\ldots,E_{b5}$	N	20000 N/mm^2	10
$E_{m1},\ldots,E_{m6}$	LN	2000 N/mm^2	20
$t_{m1},\ldots,t_{m6}$	LN	10 mm	20

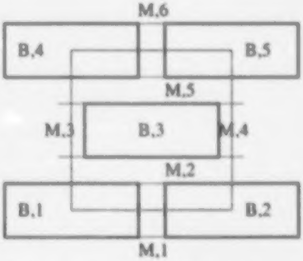

Fig. 2: Stochastic Variables

The correlation between C^1 and C^4 in fact is only a control value as it should equal the product of the other two correlation values. Taken into account the rather low amount of simulations, this requirement is approximately met, as illustrated for C_{11}:

$$\mathrm{corr}(C^1,C^2) \times \mathrm{corr}(C^1,C^3) = 0.277 \times 0.180 = 0.0498 \cong \mathrm{corr}(C^1,C^4) = 0.041. \quad \textbf{(17)}$$

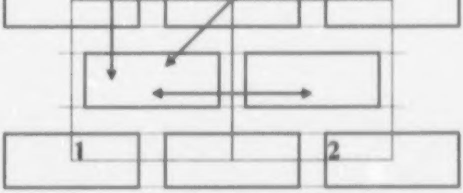

Fig. 3: Natural correlation between different basic cells

4 Simulation of correlated random fields by statistical preconditioning

As described above, the material model is defined by a number of parameters. Four of them: f_c (the compressive strength), f_t (the tensile strength), E (the initial modulus of elasticity) and E_s (the softening modulus after failure), referring to the uniaxial constitutive relationship, have been selected as the main variables. Since there is no reason to distinguish between the various areas of the masonry, a uniform random field has been assumed for these material properties.

Considering also that the four parameters are generally correlated, the corresponding fields are also correlated. The correlation between two material properties for a given basic cell has been assumed equal to 0.6 (Giannini and al. 1996). The compressive (f_c) and tensile (f_t) strength are also described by means of a continuous homogeneous random field. These continuous fields are discretised into lognormal distributed random variables. Following (Giannini and al. 1996) the correlation decay is chosen to produce a loss of correlation of 50% between neighbouring basic cells. The correlation decay of the softening modulus (E_S) is taken the same as for the initial modulus of elasticity (E). On the basis of these correlation values, the correlation function is established for the 4 material properties fitting it into the following exponential equation (Giannini and al. 1996):

$$\rho_{\Delta x, \Delta y} = e^{-(a_i \Delta x + b_i \Delta y)}, \; where \; \Delta x = abs(x_k - x_l) \; and \; \Delta y = abs(y_k - y_l) \qquad \textbf{(18)}$$

a_i and b_i (i=1,..., 4) are constant values that can be determined on the basis of the correlation of neighbouring basic cells. Their values are listed in table 3.

Table 3: Parameter values a and b for the correlation function

Material properties X_i	a_i	b_i
f_c, f_t	3.15	5.59
E, E_s	5.84	13.83

The simulation and generation of sample functions of stochastic fields can be performed in various ways (Zhang and Ellingwood 1993; Yamazaki and Shinozuka 1990; Samaras and Shinozuka 1985; Fenton and Vanmarcke 1990; Fenton 1994). The method used here to generate the stochastic fields, is a hybrid method based on covariance matrix decomposition and spectral preconditioning using a trigonometric expansion. The samples generated by this method precisely reproduce the target mean values as well as the correlation function (Yamazaki and Shinozuka 1990).

5 Finite element analysis

5.1 3x3 Rectangular shear panel

The framework as outlined in the previous sections is illustrated on a rectangular wall element. The finite element example is kept simple for purpose of illustration and computational effort. The two-dimensional shear panel, figure 4, consists of 9 (3x3) homogenised elements.

As every homogenised element (basic cell) has following dimensions: width x height = 0.220 x 0.124 m, the overall dimensions of the rectangular wall are: width x height = 0.660 x 0.372 m. Due to the homogenisation technique, realistic dimensions are obtained with a low amount of elements. Due to restrictions in the used finite element code Sbeta (Cervanka 1996) – only 4-noded iso-parametric elements are provided - the definition of the full stiffness matrix [C] is reduced to two coefficients (E,ν). As the coefficient of Poisson is taken 0.15 (Lourenço 1996), the modulus of elasticity is back-calculated from C_{11}, eq. 16.

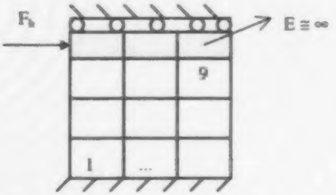

Fig. 4: FE-mesh of the 3x3 rectangular wall element

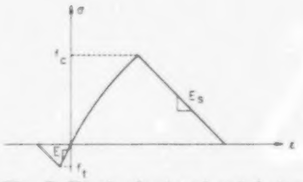

Fig. 5: Equivalent uni-axial stress-strain diagram adopted for masonry

5.2 Material model

The evaluation of the probability of collapse or the global probability of failure of a structural element, requires a mechanical model capable of describing the complex behaviour of the structure under increasing external forces and up to its failure (Giannini and al. 1996). The material model should account for the highly non-linear response of the masonry material, allowing for compression failure and tensile cracking.

Keeping in mind these requirements as well as the critical need for computational efficiency, the material model for the concrete implemented in Sbeta finite element code (Cervanka 1996; Giannini and al. 1996) has been selected. Such a model is based on a general formulation that couples an elastic-plastic constitutive relationship for the compressive behaviour, with a smeared cracking approach for the tensile failure, figure 5. The finite element code is developed for materials with brittle failure, typical for concrete. Although there are some limitations, it has been adopted for this example. As a more accurate model becomes available, it can easily be plugged into the framework, as the reliability analysis is, at least partly, independent of the finite element analysis (Faravelli 1996). For a full description of the material model, the reader is referred to (Cervanka 1996).

5.3 Modelling defects

A binomial random generator with parameter p = 0.05 is assumed to generate the location of the defects. This Binomial process generates the list of elements suffering from a defect. The expected value will be 0.05 or 5% of the total surface. Thus on average, 0.45 elements of the rectangular wall element will contain a localised defect. A uniformly distributed variable χ_i describes the reduction factors of the element *i* containing a defect. This is a vector containing 4 reduction values for each defect, referring to the 4 main material properties of the masonry: f_c, f_t, E and E_S. χ_i Is uniformly distributed in the range 0.7-0.9, meaning a reduction of 70 till 90% (Giannini and al. 1996) of the original stiffness/strength of the element. The reduction values χ_i are assumed non-correlated.

6 Application to rectangular shear panel

The random variables and their parameters adopted for the analysis are summarized in table 4. As to the response surface, the lack of fit λ is equal to: 1.23. With the outcome of the experiments, the regression coefficients are calculated, using eq. 9. The 15 coefficients θ_k read: [42.14;7.61;-0.74;-0.27;1.72;-2.32;-3.21;-2.19;1.51;3.49;-1.46;0.19;1.61;-0.11;1.14]. An analysis of the variance (ANOVA) (Montgomery 1997) is used to calculate the standard deviation of the error terms. These equal: $\sigma(\varepsilon_0)$=14.8 kN and $\sigma(\varepsilon_L)$ = 17.8 kN. To calculate the reliability or probability of failure, use was made of a First Order Reliability Method (FORM). This value is updated with an Asymptotic Importance Sampling Monte Carlo simulation (Melchers 1987; Schueremans 1998), accounting for the non-linearity of the response surface.

Table 4: Distinction between X_1 and X_2 variables and their values

Reliability analysis	**PDF**	**Mean**	**StDev**	**Cov (%)**
1. F_h: horizontal force	N	Parameter, fig.6	Variable	20
X_1=[fc,ft,E,Es]	**Pdf**	**Mean**	**StDev**	**Cov (%)**
2. f_c:(v_1) compressive strength	LN	2.62 N/mm^2(*)	0.65 N/mm^2(*)	25
3. f_t: (v_2) tensile strength	LN	0.13 N/mm^2	0.057 N/mm^2	44.1
4. E: (v_3) modulus of elasticity	LN	12020 N/mm^2(**)	708 N/mm^2 (**)	5.89
5. E_S: (v_4) softening modulus	LN	1503 N/mm^2(***)	134 N/mm^2(***)	5.89
X_2	**Parameters**			
6. Random field description;	$w_i(\xi,\eta)$			
7. (p, χ): defects	p = 0.05 and χ: 0.7-0.9			

Legend: (*) (Vermeltoort 1991; Lourenço 1996), (**) Elastic modulus calculated from C_{11}, eq. 17, (***) Softening modulus: E_s = 0.125xE (Cervanka 1996)

Once the response surface is established, the reliability analysis can be performed for different values of the horizontal force, without repeating the finite element analysis. Figure 6 shows the evolution of the reliability index/probability of failure in function of the mean value of the horizontal force. The coefficient of variation remains constant. Table 5 lists the sensitivity towards the major variables.

The main conclusions from the results in table 5 and figure 6 is that the error term has a major contribution to the total variability. This is mainly due to the localized defects (Giannini and al. 1996). For increasing values of the horizontal force, the reliability decreases from 2.2 down to 0.70. The results of the Importance Sampling Monte Carlo analysis, based on 1000 simulations, account for the non-linearity of the Response Surface.

7 Conclusions

This paper deals with many aspects of the global reliability assessment of historical masonry structures. The Response Surface method is used to estimate the horizontal failure load, based on four main variables.

Table 5: Sensitivities of random variables

Random Variable	Sensitivity
f_c	0.5014
f_t	0.0147
E	0.1860
E_S	0.1949
F_h	0.1100
ε	0.8147

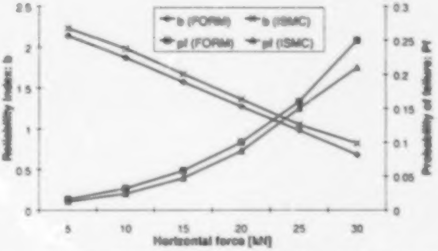

Fig. 6: Reliability index β/probability of failure P_f versus the horizontal force F_H

To reduce the amount of variables, the random variables **X** were split into two sub-vectors: $\mathbf{X}_1$ and $\mathbf{X}_2$. To account for the spatial variability of the material properties, continuous random fields were discretized in the Gauss integration points. The correlation decay of the stiffness coefficients could be calculated from the natural correlation between neighbouring basic cells. A non-linear finite element analysis was used to calculate the failure load of the shear-panels. Once the response surface is established, the reliability analysis can be repeated for different values of the horizontal load, as it is mainly independent of the calculated response surface.

It is believed that this paper illustrates the main difficulties and possibilities of a probabilistic approach to evaluate the safety of existing masonry structures. Further research will focus on the restrictions that have been made in this framework.

8 References

Andreaus U. (1996) A Failure Criteria for Masonry Panels under in plane loading, *Journal of Structural Engineering*, Vol.122, No. 1, pp. 37-46.

Braley, Matlab Reference Manual, Statistics Toolbox, 1997.

Breitung K. and Faravelli Lucia, (1996) A Response Surface Methods and Asymptotic Approximations, *CRC Mathematical Modelling Series: A Mathematical Models for Structural Reliability Analysis*, 368 pages.

Cervenka Consulting, Sbeta: computer program for nonlinear finite element analysis of reinforced concrete structures in plane stress-state, *Reference manual, Praque*, 1996.

Ellingwood Bruce R.(1996) Reliability-based condition assessment and LFRD for Existing Structures, *Structural Safety*, Vol. 18, no.2/3,pp.67-80, 1996.

Elms David G. (1997) *A Risk balancing in structural problems@*, Structural Safety, Vol. 19, no.1, pp. 67-77.

Faravelli Lucia (1989) A Response-Surface Approach for Reliability Analysis, *Journal of Engineering Mechanics*, Vol. 115, No. 2, pp. 2673-2781.

Fenton Gordon A. and Vanmarcke Erick H. (1990) Simulation of Random Fields via Local Average Subdivision, *Journal of Engineering Mechanics*, Vol. 118, No. 8.

Fenton Gordon A.(1996)Error Evaluation of Three Random Field Generators, *Journal of Engineering Mechanics*, Vol. 120, no. 12, pp. 2478-2497.

Giannini R., Pagnoni T. and Pinto P.E. (1996) A Risk analysis of a medieval tower before and after strengthening, *Structural Safety*, Vol. 18, No. 2/3, pp.81-100.

Huyse L., Maes M. A., A Methods for response-based design criteria, *Proceedings of 7th IFIP WG7.5 working conference on reliability and optimization of Structural Systems*, 1996, pp.181-187.

Huye L. and Maes M.A. (1998) Stochastic Aspects of Homogenization in Structural Analysis, *submitted to Canadian Journal of Civil Engineering*".

Liu Pei-Ling and Der Kiureghian Armen (1989) A Optimization Algorithms for Structural Reliability analysis, *Report no. UCB/SESM-86/09*, Department of Civil engineering, University of California, Berkeley, California.

Lourenço P.B., Computational Strategies for Masonry Structure", *Doctoral thesis*, TU Delft, Netherlands, 1996, 206 pages.

Melchers R. E. (1987) Structural Reliability: Analysis and Prediction, *Ellis Horwood Series in Civil Engineering*.

Montgomery Douglas (1996) *Design and analysis of experiments*, Arizona state University, John Wiley and Sons, 706 pg.

Petersen R. G. (1985) *Design and analysis of experiments*, Marcel Dekker Inc. New York, N.Y.

Samara Elias and Shinozuka Masanobu (1985) ARMA representation of Random Processes, *Journal of Engineering Mechanics*, Vol. 111, No. 3, pp. 449-461.

Schueremans L. (1998) Reliability analysis of Structural Unreinforced Masonry, *research report as visiting researcher at the University of Calgary*, Alberta, Canada, K.U.Leuven, Belgium, 108 pages.

Veneziano D., Casciatti F. and Faravelli L. (1983) Method of Seismic fragility for complicated Systems, *Proceedings of the 2nd Committee on safety of Nuclear Installations*, Specialistic meeting on probabilistic Methods in Seismic Risk Assessment for NPP, Livorno, CA.

Vermeltfoort A. Th. and Raijmakers T.M.J. (1991) Vervormingsgestuurde mesoschuifproeven op metselwerk, *TNO Rapport B-92-1156*, TNO Delft, The Netherlands.

Yamazaki Fiumo and Shinozuka Masanobu, (1990) Simulation of Stochastic Fields by Statistical Preconditioning, *Journal of Engineering Mechanics*, Vol. 116, No. 7, pp. 268-287.

Yoa Timothy K. J. and Wen Y.K. (1996) Response Surface method for time-variant reliability analysis, *Journal of Structural Engineering*, Vol. 122, No. 2, pp.193-201

Zhang Jung and Ellingwood Bruce (1994) A Orthogonal Expansions of Random Fields in Reliability Analysis, *Journal of Engineering Mechanics*, Vol. 120, No. 12, pp. 2660-2677.

125 DURACRETE: SERVICE LIFE DESIGN FOR CONCRETE STRUCTURES

T. Siemes and C. Edvardsen

[1]TNO Building and Construction Research, The Netherlands, [2]Consulting Engineers and Planners AS, Lyngby, Denmark

Abstract

In the past decades much effort has been put into the improvement of the durability of concrete structures. This has resulted in a reasonable understanding of the main degradation processes or in experience with measures to prevent degradation. The results of this effort can be found in the present concrete codes and in manuals on durability design. The design rules are in general presented as deem-to-satisfy rules. If the rules are followed it may be assumed that the structure is durable. The present approach does not give direct insight in the service life, the necessary maintenance or the probability of premature failure. Further it is clear that lack of durability can have an influence on the structural behaviour. The direct relationship between durability and safety and serviceability of concrete structures has however not been made in the concrete codes. In the Brite-EuRam project 'DuraCrete' the durability design has been developed into a service life design based on performances and on reliability for reinforced concrete structures. This offers the possibility to present the design on the same level as the structural design that has also been based on performances and reliability. The structural and service life design can even be integrated. The 'DuraCrete' approach offers good opportunities for the service life design of other structural materials and building materials.

Keywords: durability, probabilistic design, reinforced concrete

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1 State-of-the-art durability design of concrete structures

The present design approach with respect to durability of concrete structures is based on a reasonable understanding of the main degradation processes for concrete, reinforcement and prestressing steel. The performance of the design is however not explicitly formulated as a service life. It is based on deem-to-satisfy rules (e.g. minimum cover, maximum water/binder ratio, and crack width limitation) and the assumption that if these rules are met, the structure will achieve an acceptable long but unspecified life. The information about the service life to be achieved is to a large extent empirical. Improving the durability increases building costs without any quantification of the reduction of maintenance costs or failure costs. Current design methods only permit to calculate the whole life cycle costs from assumptions with respect to maintenance and failure rates. There are thus no objective means for demonstrating that future maintenance and repair costs will be low.

This common design approach to durability has other disadvantages. The rules are inadequate in some aggressive environments, while they are too rigorous in other environments. In some cases, this results in a 'belts and braces' approach (many different types of measures on top of each other) which may contain unnecessary and even counteractive measures.

Lack of durability can cause serious safety and serviceability problems for structures. Despite this, usually designers have considerably more attention for load and resistance based structural design than for durability design. Recent history has however shown that due to a lack of durability, serious collapses and other types of damages may occur with large amounts of damage.

On 4 December 1984 at 7 o'clock in the morning the Ynys-y-Gwas bridge in the neighbourhood of Port Talbot in Wales (UK) collapsed (Woodward 1988). This concrete bridge was a single span segmental post-tensioned structure. Its span was about 18 meters. The bridge was constructed in 1953 and carried a minor road. When its deck collapsed there was no traffic on the bridge. The cause of the collapse was serious corrosion of the post-tensioned tendons. The corrosion took place at the transverse joints in which chloride-containing water could penetrate. This penetration was possible as a consequence of a number of factors, such as the lack of a slab over the beams, ineffective waterproofing, inadequate protection of the tendons, opening of the gap between the segments under live load, poor workmanship and the damp environment over the river.

In Berlin the southern outer roof of the Congress Hall collapsed on 21 May 1980 due to hydrogen-induced stress corrosion (Schaich 1980). One person died and another was badly injured. The outer roof was characterised by a reinforced concrete arch, which was hollow. Post tensioned steel tendons, which were installed in roof sheeting served as the anchorage of the arch against an inner ring beam. In a part of the roof the sheeting broke away from the ring beam. In another part the sheeting broke away at the outer arch. Corrosion could occur due to design faults leading to cracks in the joints in combination with a porous mortar in the prestressing ducts.

In 1990 a reinforced concrete gallery in Wormerveer (NL) collapsed. This was caused by chloride induced corrosion. Due to poor construction, the main reinforcement in the gallery was situated in the lower part (the actual compression zone) instead of the upper part of the slab (the actual tension zone). This resulted in a crack where de-icing salts could penetrate. In Melle (B) a prestressed concrete bridge collapsed during the passage of a tank-car filled with gas. The driver died. The cause was a crack that opened during the passage of high loads. Chloride could penetrate through this crack and reach the post-tensioned tendons.

The previous list of structures in which serious accidents occurred as a consequence of insufficient durability demonstrates that design for durability can be as important as common structural design. These failures are partly due to bad construction and workmanship in addition to bad joint design. These examples are presented here to demonstrate that lack of durability may lead to collapse. Lack of durability may also lead to other types of failures, leading to a reduced serviceability of the structure and to premature maintenance, repair or even demolition.

In the last decades much effort has been put into the development of models and methods for predicting deterioration of concrete structures. A number of methods have matured to a level where these can be used in a formalised approach for the assessment and design of concrete structures with respect to destructive mechanisms.

Controlling the durability of concrete structures will be a fundamental challenge for the engineer in the next millennium. Past decades have shown us that the classical procedures for design, construction and use of concrete structures have failed to provide reliable long-term performance. Deterioration processes, in particular corrosion of reinforcement, frost action, alkali aggregate reactions and sulphate attack have caused serious damage to concrete structures.

To improve this situation a new concept for durability design needs to be established. Similar to the current procedures for structural design, a design for durability should be performance based taking into account the probabilistic nature of the environmental aggressiveness, degradation processes and material properties involved.

In order to quantify the durability the concept of a service life design has been introduced. In this respect the performance requirements for a service life design as stated in the CEB-FIP Model Code (1990) has been adopted:

> *Concrete structures shall be designed, constructed and operated in such a way that, under the expected environmental influences, they maintain their safety, serviceability and acceptable appearance during an explicit or implicit period of time without requiring unforeseen high costs for maintenance and repair.*

Such a rational design for durability, however, requires both an overall methodology and calculation models for the actual degradation processes of concrete structures. Similar to the structural design code for loads, safety requirements and limit states must be defined for the design service life.

2 DuraCrete project

The new durability design methodology should be able to document the efficiency of the materials to resist the aggressiveness of typical environments in Europe. The structural designer will, therefore, be able to document the fulfillment of a specific limit state. For the designer the deterioration models showing the degradation over time, or showing the service life as a function of appropriate design parameters, are valuable tools. With the aid of this methodology for durability design the designer can make decisions on the required dimensions and material specifications for structures with service life requirements.

One consequence of the requirements to design for durability is that structural engineers and designers need to be educated in the durability aspects of various materials as well as in the structural layout, the effect of execution on the in situ qualities, and the interaction between structure and environment. This is a prerequisite to enhance cost optimal durability designs incorporated into the overall 'life cycle' costs, and is thus a prerequisite to be able to satisfy the future requirements to the performance of concrete structures. An immediate spin-off will also have to be reflected in the structural engineering curriculum. The design engineer will need a design guide to assist him in making a proper service life design, not only for new structures but also for the redesign of existing concrete structures. A first edition of such a guide has been developed in the project 'DuraCrete'. The project has been subdivided in 8 separate tasks:

1. Establishing theoretical framework for Task 7 and providing a mini-project to show how all the tasks are interrelated.
2. Modelling of the deterioration mechanisms. A simple form of each of the models is provided for engineering design purposes.
3. Evaluation of compliance tests available for the key-point parameters of the models selected in Task 2. Furthermore, a number of test results for different concrete mixes are provided with regard to carbonation resistance, resistance against chloride penetration, corrosion resistance, frost de-icing salt resistance and finally fatigue.
4. Statistical quantification of the uncertain parameters regarding the deterioration models. The results are tables identifying statistical distributions, mean values and coefficients of variation (standard deviations). Furthermore the environmental parameters are classified and quantified and the mechanical behaviour of concrete structures is statistically quantified.
5. Provides Task 6 with a range of relevant structural elements designed using current code recommendations in Europe.
6. Limit state functions, sensitivity measures for the input parameters and finally target reliabilities corresponding to today's practice concerning durability (Task 5 structural elements).
7. Developing General Guidelines for Durability Design and Redesign (in short: A Durability Design Guide) based on the work carried out in the previous tasks of the DuraCrete project:
8. Dissemination.

3 Framework

3.1 Basis for design

In principle, the design strategy for durability is to select an optimal material composition and structural detailing to reliably resist, for a specified period of use, the degradation threatening the structure. In the case of redesign there is however less freedom available, as we have to accept (partly) the choices that have been made during the original design. As the redesign is based on the same principles as the design we will concentrate further just on the design.

Sometimes, it will be clear that a design is very reliable (the structure is protected from the aggressive environment by tanking, membranes, coatings etc.; non-reactive or inert, materials like stainless steel reinforcement has been used; the aggressive reactions are inhibited by cathodic protection). In that case a further prove of the reliability may be omitted.

The durability design strategy has different types of interventions. For example, corrosion protection could be achieved by selection of appropriate cover and concrete mix. In addition, the structure can be made more robust against aggressive environments of different sorts by appropriate detailing such as minimising the exposed surface, by rounded corners, or by adequate drainage.

The durability design will be based upon:

- realistic and sufficiently accurate definitions of environmental actions (different micro-environmental aggressiveness classes) depending on the resulting type of degradation
- material parameters for concrete and reinforcement
- mathematical models for degradation processes. In the present version of the guide the mechanisms which have been modelled have been limited to:
  - carbonation induced corrosion
  - chloride induced corrosion
  - cracking and spalling of concrete due to corrosion products.

3.2 Structural design

The durability design guide follows the same principles (reliability and performance) as a structural design code. As is the case of the structural design load, resistance variables also need to be defined. By definition a load variable is a variable where an increase of the numerical value leads to a reduction of the reliability of the structure, whereas an increase of the numerical value of a resistance variable leads to an increase of the reliability.

The basis of the conventional design procedure for the safety and serviceability of structures with a static loading can be expressed as the limit state function. This limit state defines the border between an adverse state (such as collapse, buckling, deflection, and vibration) and the desired state. The limit state can in principle be formulated as:

$$R - S = R(X_1, X_2, \ldots, X_n) - S(X_{n+1}, X_{n+2}, \ldots, X_m) = 0 \qquad (1)$$

in which:

R - a function that describes the load bearing capacity of the structure
S - a function that describes the influence of the load on the structure
X_i - a basic variable for the functions R or S.

The structural design procedure is worked out in such a way that the failure probability is restricted:

$$P\{\text{failure}\} = P_f = P\{R - S < 0\} < P_{\text{target}} = \Phi(-\beta) \qquad (2)$$

in which:

P{failure} or P_f - the probability of failure of the structure
$1 - P_f$ - the reliability of the structure
P_{target} - the accepted maximum value of the probability of failure
Φ - standard normal distribution function
β - reliability index (this parameter is normally used in codes in stead of the failure probability)

With the aid of probabilistic techniques this failure probability can be calculated. In practice the design has been simplified to a semi-probabilistic level with characteristic values and partial factors γ. These values are calibrated in such a way that the target reliability will be achieved:

$$R_c / \gamma_R - S_c \cdot \gamma_S = R_d - S_d > 0 \qquad (3)$$

in which:

R_c - load-bearing capacity of the structure based on characteristic values
γ_R - material factor
S_c - characteristic value of the influence of the loading
γ_S - load factor.
R_d - design value of the load bearing capacity
S_d - design value of the load.

3.3 Time dependent design

In the performance based structural design both the resistance R and the load S are considered to be time independent. In many loading situations this is not realistic. Relationship (1) should then be rewritten as a time dependent limit state function (Siemes and Rostam 1996):

$$R(t) - S(t) > 0 \qquad (4)$$

A special case for this limit state function occurs if either R or S is not time dependent. The working out of these relationships do in principle not differ from working out (2).

Relationship (4) applies for all t in the time interval (0,T) where T is the intended service period (i.e. reference period). Even if the loading or the capacity of a structure is time dependent, the limit state functions for the design of structures are never formulated in this way (with an exception for fatigue). They are always simplified to time independent quantities for which relationship (1) can be used. From a mathematical point of view it can be stated that relationship (4) is of the same type as the functions that will have to be solved for the durability design. The service life concept can be expressed in a design formula, equivalent to relationship (2):

$$P_{f,T} = P\{R - S < 0\}_T < P_{target} = \Phi(-\beta) \qquad \textbf{(5)}$$

in which:

$P_{f,T}$ - the probability of failure of the structure within T
T - intended service period.

It will probably be possible to simplify relationship (5) to a similar one as for the conventional design procedure (3), but at a later stage.

The mathematical model for describing the event "failure", i.e. passing a durability limit state, comprises a load variable S and a resistance variable R, (Figure 1). Failure occurs if the resistance is smaller than the load. The probability of failure within the period of time [0, T], $P_f(T)$ is defined as the probability of the load not exceeding the resistance within the given period. I.e.,:

$$P_f(T) = 1 - P\left\{R(t) > S(t) \quad \forall t \in [0;T]\right\} \qquad \textbf{(6)}$$

As either the resistance R or the load S or both can be time dependant quantities, the failure probability is a time dependant quantity.

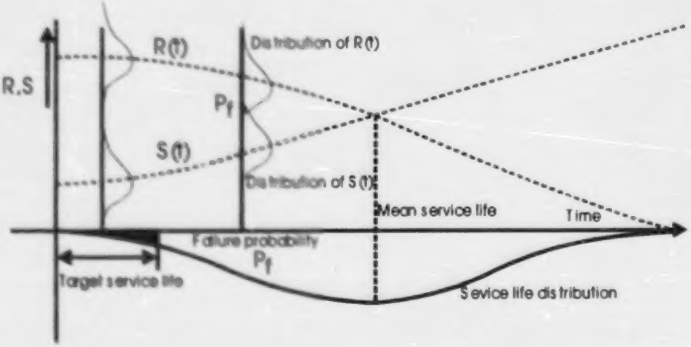

Fig. 1: Failure probability and target service life (illustrative presentation)

With equation (6) a target service life t = target can be determined for a certain allowable failure probability, thus satisfying a certain acceptance criterion. As in structural design, the failure probability, P_f, and thus the acceptance criterion, is usually being given in terms of a safety index or reliability index β, defined by

$$\beta = -\Phi^{-1}\left(P_f\right) \tag{7}$$

3.4 Limit states and design service life

The first step of this durability design will be the definition of the desired/required performance of the structure. The client or the owner of the structure is asked to define their requirements for quality and target service life. The definition of performance criteria will be related to a limit state criterion. Figure 2 shows in principle the performance (damage function) of a concrete structure with respect to reinforcement corrosion and related limit states. In general points 1 and 2 represent serviceability limit states related to durability and point 3 and 4 represent ultimate limit states where point 3 relates to both durability and safety.

In the DuraCrete project the following durability limit states have been identified with regard to service life:

1. Depassivation of reinforcement.
2. Cracking of concrete cover
3. Spalling of concrete cover
4. Collapse.

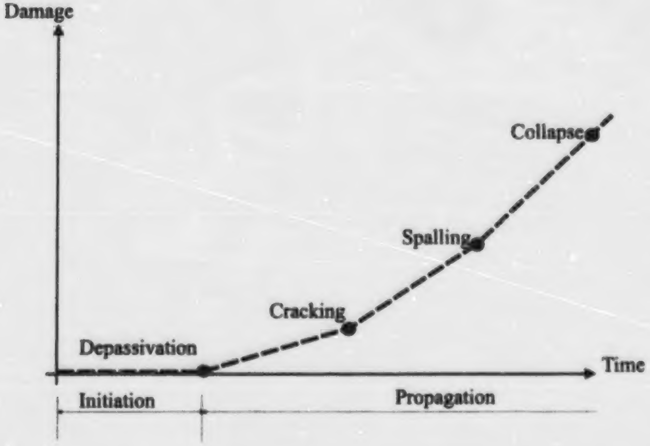

Fig. 2: Determination of service life and limit states with respect to reinforcement corrosion

3.5 Degradation models

The second step of the durability design is to analyse the environmental actions and to identify the relevant degradation mechanisms. Mathematical models describing the time dependant degradation processes and the material resistance against it are needed. The big step forward to performance related durability design is that these models enable the designer to evaluate the time-related changes in materials and structures depending on the specific material and environmental conditions.

The different models used for this durability design consist of design parameters such as structural dimensions, environmental parameters and material properties that correspond to the load and resistance variables of the structural design procedure. In the case of chloride induced corrosion the following model describing the initiation of corrosion has been identified:

$$x(t) = 2 \cdot k \cdot \sqrt{D_{cl,0} \cdot k_e \cdot k_c \cdot t \cdot \left(\frac{t_0}{t}\right)^n} \qquad (8)$$

$$k = \mathrm{erf}^{-1}\left(1 - \frac{C_{cr}}{C_s}\right) \qquad (9)$$

Table 1: Load and resistance variables in relationship

Load variable	**Resistance variable**
C_s **= chloride surface concentration**	$x(t)$ **= required concrete cover**
$D = D_{cl,0} k_e k_c \left(\frac{t_0}{t}\right)^n$ **= chloride diffusion coefficient**	C_{cr} **= critical chloride concentration**

The environmental factor k_e (accounting for the actual environmental class), the curing factor k_c (accounting for the actual curing period) and the age factor n (accounting for the desired service life t) are neutral factors, i.e. neither load nor resistance factors. These factors are used to determine the actual, effective chloride diffusion coefficient

Taking the example of onset of chloride induced corrosion the durability inputs are as follows:

The limit state is given by the requirement that the chloride concentration at the surface of the reinforcement may not reach the critical chloride concentration.

The resistance R is given by the critical chloride concentration and the quality and thickness of the concrete cover.

The load S is represented by the actual chloride concentration at the reinforcement level. This depends on material parameters (chloride diffusion coefficient) and environmental effects.

3.6 Level of reliability and quality control

The knowledge about the structural and material characteristics of a structure is uncertain due to the uncertainties associated with structural layout, material characteristics, execution and environmental input. Testing and quality control is therefore important at several stages during the life of a structure. To check the real performance of the structure the material properties, the geometrical and execution variables that influence the selected degradation models have to be affected by quality control. The aim is to define and to update the statistical quantities (type of distribution, mean value, standard deviation) of these affected stochastic variables.

If a high degree of quality control can be documented and quantified it serves as basis for documenting service lives. In this way the updating of the statistical quantities by reducing the coefficient of variation will result in the possibility to consider lower partiai factors. The combined set of problems to be considered can be summarised as follows:

1. The factual information on the aggressiveness of the environment, the characteristics of the materials in the structure, and the structure-environment interaction, is uncertain.
2. The degree of uncertainty depends on the type and direct relevance of the available information on the above variables.
3. The reliability of the available information is different depending on the stage being considered, such as the design stage, the construction stage, the handing over stage, and the period of use.
4. From a durability design point of view this means that design input data with different levels of reliability must be used at the different stages.

Quality control (QC) in the sense of the durability design guide is mainly to control the variation of the material parameters and the geometrical variables at the different stages. For concrete the quality control can be based on cement or concrete properties on basis of compliance tests to measure the relevant quality of the cement or the concrete under standardised conditions. The test results may serve as input for the designer in the design situation. The cement producer can classify the potential material resistance against degradation, measuring the following quality controlled material parameters:

- Carbonation: carbonation penetration coefficient D_{eff}
- Chloride ingress: chloride diffusion coefficient, D_o
- Corrosion rate: electrical resistivity, ρ_o.

Compliance tests to measure the potential quality of the supplied concrete with the composition defined (binder content, water/cement ratio) in the design by the contractor/ready mix producer under standardised conditions. The same QC-measurements like for the cement are performed. By means of this the quality of the concrete supplied can be checked against the design requirements.

Similar to the classification system used in structural design (for example the concrete strength classes) the durability design can be based on durability classes of concrete which corresponds to specified values (characteristic values) with regard to the carbonation penetration rate, the chloride diffusion coefficient and the electric resistivity.

Verification tests are needed to determine the actual quality of the concrete in the structure in service. The actual resistance of the concrete to premature degradation is influenced by execution (compaction, curing) and micro-environment.

3.7 Further developments for DuraCrete

To demonstrate the use of the DuraCrete design method a practical example for the lining of a bored tunnel has been presented in (Breitenbücher *et al.* 1999). Although it has been possible to make the service life design for this lining and also for all the other parts of the tunnel, it is obvious that further developments have to be made. Some of them are:

- reliable mathematical models for degradations like frost, frost and de-icing salts, alkali aggregate reaction, sulphate attack have to be developed
- more information and data are needed for the effects of corrosion of the reinforcement (such as corrosion rate, cracking, spalling and collapse); until now most research has been directed to avoid corrosion and is therefore restricted to the initiation phase
- better understanding of the corrosion process under water and under the ground; due to lack of oxygen or to slow transport of aggressive substances the type and the rate of corrosion products may be different from atmospheric corrosion
- improved knowledge of environmental conditions
- determine the reliability of durability measures like repair materials and methods, coatings, cathodic protection etc
- knowledge must be disseminated; the method must be used in practice and in education.

The two main results of the DuraCrete projects are that all disciplines that are necessary for a durability design are brought together: material science, reliability engineering, structural engineering, and further a framework for the durability design has been presented. The objective has not been to develop new knowledge on degradation of concrete. The list of developments to be made in DuraCrete might suggest that the method is not yet applicable in practice. The example of the bored tunnel by Breintenbücher *et al.* (1999) proves the contrary.

4 Extension to other materials

4.1 Structural materials

Modern structural codes for steel, aluminium, timber, plastics and masonry are in principle based on performances. These performances are formulated as limit states with reliability indexes and reference periods. This is similar to the approach in modern concrete codes. In principle, it will therefore be possible to apply the

DuraCrete approach to these materials.

The use of reinforced and prestressed concrete has always been connected with research on degradations and durability. In this respect concrete differs from most of the other structural materials. Timber is an exception to this. This means that for most of the structural materials a lack of practical knowledge might be available. At the same time we must realise that a designer of concrete structures is always aware of the fact that he must take care of the durability of the structure.

The durability measures that are taken at the present state of knowledge are in general the application of coatings (organic and metallic), sacrificial layers (e.g. pile sheeting) or for steel cathodic protection. In severe conditions durable materials are applied such as stainless steel instead of low carbon steel.

With the aid of the DuraCrete approach it is possible to make an objective estimate about the service life costs, including the maintenance. It will be a challenge for the other materials to present a life cycle costing for structures made of other structural materials than concrete. Owners of large structures, like buildings and infrastructures will more and more demand such an objective cost comparison. Contractors working on the basis of a design and construct contract can make their profits by selecting the material with the lowest overall price.

4.2 Building materials and components in general

Besides structural materials the building industry uses a lot of other materials, such as plasters, copper and bitumen. The industry uses further structural materials in non-structural applications, like steel window frames or timber doors. This situation differs in some aspects from the structural materials:

- the design is in principle not explicitly based on performances and on reliability
- the design is based on experience and on deem to satisfy rules in codes
- for a part of the applications the service life is restricted
- in some of the applications there is a high degree of repetition.

Making a service life prediction on the bases of performances and on reliability, as in DuraCrete, will require considerable effort. On the other hand a great deal of work has already been done. CIB W80/RILEM 175-SLM, its preceding committees and other national and international committees on durability of building materials and components have through the past years obtained useful results. This applies in particular for the CIB W80/RILEM 71-PSL (1987), RILEM technical recommendation: "Prediction of service life of building materials and components".

The main shortcoming of past work is the fact that in many cases the end of the service life has not been defined on basis of performance but rather on the basis of a lack of functionality or on other subjective judgements. Furthermore, aspects of reliability are not explicitly taken into account. However, these shortcomings can easily be overcome.

An essential part of the research and testing in the past has been based on routine testing where the testing conditions are not similar to the in-service environment in which the building material or component is to function. These tests are essentially robustness tests. An example of this type of testing is the salt spray test for metals and organic coatings. Passing this test with a good result does not necessarily mean that the service life in an industrial environment with high SO2 contents in the atmosphere will be adequate.

5 Concluding remarks

The DuraCrete approach that has been developed for the service life design of new and existing concrete structures is based on the same principles and framework as the performance based structural design. It is based on terms such as performances, limit states, reliability and reference period or design service life. It may be expected that this approach will be the basis for service life design of concrete structures in the future.

The DuraCrete approach can easily be developed towards a methodology that can be used for the service life design of structural materials, like steel, aluminium, timber, plastic and so on. Defining the limit states, degradations, environmental conditions and undertaking the relevant tests are future activities that must be completed if this new design approach is to be used in practice.

For non-structural building materials it is also possible to use the new service life design approach. In fact many steps towards the application of the new methodology have already been made. In general it can be stated that the notions of performance and reliability have to be firmly incorporated in the service life prediction methods. The fact that many applications have a highly repetitive character and that the service life is often restricted (in relation to structural applications where design service lives of 100 or 200 years are sometimes required) make it easier to complete the necessary developments.

6 Acknowledgement

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7 References

BE-1347/TG7/ Report R14 (1999), "General Guidelines for Durability Design and Redesign", Brussels: Brite-EuRam, 1999, Project No. BE95-1347

Breitenbücher, R., Gehlen, C., Schiessl P., Hoonaard, J. van den, and Siemes, T. (1999), Service life design for the Western Scheldt Tunnel, Proceedings 8th International Conference on Durability of Building Materials and Components, Vancouver Canada, 1999

CEB/FIP Model Code for Concrete Structures (1978), Recommendation of FIP and CEB.

CIB W80/RILEM 71-PSL, Recommendation: Prediction of service life of building materials and components, Materials and Structures, Vol. 20, No. 115, 198

Schaich, J., Kordina, K., Engell, H.J. (1980), Partial collapse of the Congress Hall in Berlin - Causes of damage - report summary, Beton und Stahlbetonbau, Vol. 75, No. 12, 281 – 294

Schiessl, P. (ed.), Hergenröder, M., Möller, J., Nilsson L-O., and Siemes A.J.M. (1997), CEB Bulletin 238 'Durability design - Performance and reliability based design of concrete structures, Published by CEB, Laussane

Siemes, A.J.M. (1997), New developments towards a performance based durability design for concrete structures; Symposium Advanced Design of Concrete Structures, 12-14 June 1997, Götenborg; Uitg. Chalmers University S, June 1997, pp. 138 –148

Siemes, A.J.M. and Rostam, S. (1996), Durable safety and serviceability a performance based design format, IABSE report 74: Proceedings IABSE colloquium 'Basis of design and Actions on Structures Background and Application of Eurocode 1, Delft, pp. 41-50

Woodward, R.J. and Williams, F.W. (1988), Collapse of Ynys-y-Gwas bridge West Glamorgan, Proceedings of the Institution of Civil Engineers, Part 1, Volume 84, August, pp. 635-669

126 FAILURE PROBABILITY OF DETERIORATING REINFORCED CONCRETE BEAMS

B. Teplý, D. Novák, Z. Keršner, and W. Lawanwisut

Technical University of Brno, Czech Republic

Abstract

For a specific reinforced concrete beam the depassivation of reinforcement is assessed and a consequent corrosion process in time is evaluated using the available numerical models. The cross-section of a beam is analyzed using the "layer approach". Division into layers enables us the efficient modelling of the carbonation process and the consequent corrosion of reinforcement. The influence of deterioration on failure probability is assessed.

Keywords: Bayesian updating, carbonation of concrete, corrosion of reinforcement, deterioration, failure probability, importance sampling, reliability

1 Introduction

The deterioration caused by aggressive environment may have a significant impact on safety and serviceability of reinforced concrete (RC) structures. The important degradation processes are the carbonation of concrete, the ingress of chloride ions (both these effects cause the depassivation of reinforcement) and the corrosion of reinforcement. Such time-dependent processes are governed by many uncertainties - random parameters. Consequently, by these stressors the strength and stiffness of RC members can be considerably decreased. Therefore it is important to develop a suitable tool for the prediction of such processes and to utilize them while investigating the reliability or the repair strategy of structural members.

The problem of deteriorating RC beams have gained recently a strong attention of several researchers, Lee et al. (1996, 1998): A nonlinear finite element (NL-FE) solution of a simple RC structure with the rebar corrosion effect is investigated by a

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deterministic approach. The deterioration is considered to be mainly due to the degree of corrosion and a consequent decrease of steel yield point and bond performance. Constitutive laws for these effects are based on experiments (which unfortunately cannot be generalized). Boer and Veen (1998) also presented a NL-FE approach, once again deterministic. No models for material deterioration were involved - the crack analysis of concrete and the reduction of the bar diameter are not explained in detail. Stewart and Rosowsky (1998) has developed an advanced approach taking into account the random nature of parameters involved and have presented a preliminary probabilistic framework for a time-dependent life cycle analysis for bridge slabs. The reduction of cross-sectional areas of steel bars and both the transverse and longitudinal cracking of concrete were taken into account.

The present paper describes a probabilistic approach for the reliability investigation and the life-time prediction of simply supported RC beams including deterioration in time. Only the ultimate limit state is controlled and the depassivation of reinforcing rebars is supposed to be caused by the carbonation of concrete. The presence of chloride ions (Cl^-) is excluded in this case. The physical, chemical, technological and environmental characteristics are considered to be random variables within the framework of a fully probabilistic approach.

If the bending action prevails and the attention is paid to the ultimate limit state of simply supported beams only, the limit state function may be represented by comparing the ultimate bending moment (resistance) and load action in one cross-section. First, the carbonation depth as a main factor influencing the initiation of corrosion of reinforcement bars is treated more precisely using the so-called Bayesian updating method based on short-term in-situ measurements (measured carbonation depth statistics). Secondly the appropriate advanced simulation techniques (importance sampling) are used for the probabilistic prediction of deterioration in time measured by the theoretical probability of failure p_f. Using p_f -time curves and "isoprobability" curves the residual life-time of the structure can be predicted.

2 Deterioration processes of reinforced concrete

2.1 Carbonation of concrete

The model of time-dependent carbonation depth x_c of ordinary Portland cement (OPC) concrete by Papadakis et al. (1992) is used. This computational model is based on the mass conservation of CO_2, $Ca(OH)_2$ and CSH (hydrated calcium silicate) in any control volume of the concrete mass. The simplified carbonation depth formula for OPC concrete is expressed here as:

$$x_c = 0.35\rho_c \frac{\left(\frac{w}{c}-0.3\right)}{\left(1+\frac{\rho_c w}{1000c}\right)} f(RH) \sqrt{\left(1+\frac{\rho_c w}{1000c}+\frac{\rho_c a}{\rho_a c}\right) c_{CO_2} \frac{22.4}{44} 10^6 t} \qquad (1)$$

where x_c is the carbonation depth (mm) for time t (years), ρ_c, ρ_a are the mass density (kg/m^3) of cement and aggregates, resp., w/c, a/c are the water/cement, aggregate/cement ratio, resp., *RH* is the ambient relative humidity, c_{CO_2} is CO_2 content in atmosphere (mg/m^3), a is the aggregate unit content. In the original formulation (Papadakis et al. 1992) the model for high values of *RH* does not provide satisfactory results. This has been overcome by implementing the step-wise linear relationship *f(RH)* extracted from experiments reported by Matoušek (1997), see also Novák et al. (1996).

A total of 11 variables (see details in Table 1) are involved in this model, and in some cases not all appropriate information is available. Therefore, a simpler model may be useful. For this purpose the model by Bob (1996) seems to be appropriate:

$$x_c = 150\frac{Ckd}{f_c}\sqrt{t} \tag{2}$$

where x_c is the average depth of carbonation (or chloride penetration) (mm), f_c is concrete compressive strength (MPa), t is time of CO_2 or/and Cl^- action (years), C is the coefficient of cement type, k introduces the influence of humidity (environmental conditions) and d is coefficient for CO_2 content. Both models were randomized and compared - see Keršner et al. (1996). In these forms the models are utilized in the present paper. The purpose of modelling the carbonation depth is basically to calculate the time at which depth x_c reaches the concrete cover c (initiation time T_i).

2.2 Corrosion of reinforcement

An accurate and reasonably general computational model for corrosion of depassivated steel bars embedded in concrete is still missing. Note that both the uniform and pitting type of corrosion should be modelled. Presently the models used e.g. by Andrade et al. (1996); Rodriguez et al. (1996) and Stewart and Rosowsky (1998) seems to be sufficient for the prediction of uniform corrosion. The formula for the time related rebar diameter decrease is as follows:

$$D(t)=\begin{cases} D_i & t \le T_i \\ D_i - 0.116 i_{corr}\alpha(t-T_i) & T_i < t \le T_i + (D_i/0.116 i_{corr}\alpha) \\ 0 & t > T_i + (D_i/0.116 i_{corr}\alpha) \end{cases} \tag{3}$$

where D_i is the initial bar diameter (mm), T_i is the time to initiation (years) (i.e. depassivation) and i_{corr} is the current density (normally expressed in $\mu A/cm^2$). Parameter α expresses the type of corrosion. For homogenous corrosion α is equal 2, however, when a localized corrosion (pitting) occurs α may reach values up to 8 (see Rodriguez et al. 1996).

3 Computational model

3.1 Nonlinear analysis

The codes give approximate expressions for computing the ultimate flexural resistance of reinforced concrete sections. In order to make the computation more accurate a simple method that satisfies equilibrium and compatibility is utilized. The reinforced concrete section is divided into layers, each layer is considered to be in a state of uniaxial tension or compression. Maximum strains are considered initially, then stresses in layers (both concrete and reinforcement) are computed using the idealized stress-strain diagrams given in Fig. 1. By iterative algorithm equilibrium is checked resulting in an ultimate bending moment M_u. This "layered" approach is also suitable for modelling the deterioration.

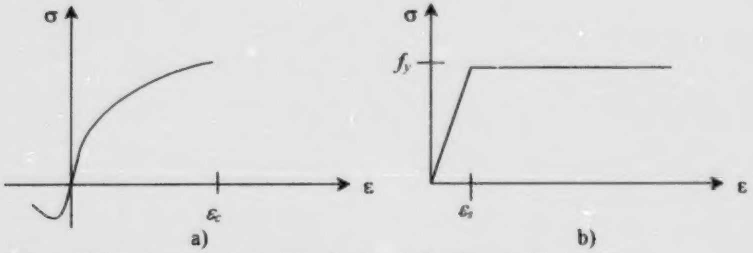

Fig. 1: Stress-strain diagrams: a) concrete; b) reinforcing steel.

3.2 Limit state function

For a reliability analysis a function $g(X)$ of basic random variables $X = X_1, X_2, \ldots, X_m$ is defined comparing ultimate bending moment M_u (resistance) with load bending moment M (action of load). Limit state function is then given as:

$$g(X,t) = \varnothing M_u(X,t) - M \tag{4}$$

and is a function of time t, model uncertainty factor $\varnothing$ and m random variables. Note that the decrease of $M_u(X,t)$ is strong when $t > T_i$. A beam is considered to be safe if:

$$g(X,t) > 0 \tag{5}$$

For the purpose of the present study the load action M is taken as a deterministic value. This arrangement enables us to predict the structural life-time of deteriorating structure for a prescribed load level.

4 Reliability analysis

The material of concrete and reinforcing steel deteriorate in time and consequently the ultimate bending moment decreases. The goal is to quantify the influence of deterioration on the reliability. The failure probability is defined as:

$$p_f = P(g(X,t) \leq 0) \tag{6}$$

and can be evaluated at several time points t_i ($i = 1, 2, \ldots, n$). The increase of failure probability in time can be estimated here by advanced simulation techniques e.g. importance sampling (Bourgund and Bucher, 1986).

5 Numerical example – case study

5.1 General remarks

A particular storage building structure is composed of prefabricated RC units. The study is focused on the roof beam where the carbonation depth was investigated in-situ in several positions (the age being 11 years). The cross-section of the beam is shown in Fig. 2.

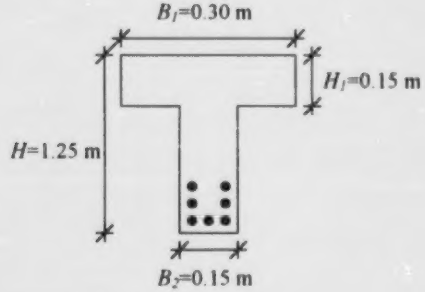

Fig. 2: Critical cross-section with reinforcing bars.

5.2 Random variables

In-situ investigation also provided other values of several input variables and their statistical parameters; some other parameters were gained by engineering judgement. All of them are summarized in Table 1.

5.3 Results of parametric study

The approach described above was used and the results of the probabilistic analysis are as follows:

The probability of failure is obviously increasing in the course of time due to corrosion of reinforcement. This effect can be observed in Fig. 3 for different levels of load (where the p_f- time curves are shown). Similar results are depicted in a different way by the mean of the isoprobabilities (i.e. curves of constant probability) - see Fig. 4. The p_f-time curves or the isoprobabilities can be used for the structural life-time (L) prediction. It could be observed in Fig. 4 e.g. for M =550 kNm: If the admissible p_f would be 10^{-5} then $L \approx 75$ years; for $p_f = 10^{-6}$ $L \approx 25$ years only.

The in-situ measurements of carbonation depth x_c after 11 years of the utilization of the building were performed. The results of the mean value of 6 mm and standard deviation of 1.2 mm were obtained, and the Bayesian updating (see e.g. Novák et al. 1996) of the x_c evaluation in time could be assessed. Based on this new x_c-t function also an updated p_f-t curve could be computed. The comparison of a prior and posterior p_f-time curves is shown in Fig. 5, the latter curve being more accurate as the utilization of in-situ measurements takes into account the effect of local conditions. This cannot be accounted for by a general analytical model itself. Such updated p_f-time curves make it possible to gain a more reliable prediction of the structural life-time.

Table 1: Basic random variables.

Variable	Symb.	Unit	Mean	COV (%)	PDF
Uncertainty factor of model	$\varnothing$	-	1	0.1	N
Ambient CO$_2$ content	c_{CO_2}	mg/m^3	800	0.15	N
Relative humidity	RH	-	0.85	0.1	N
Unit content of cement	c	kg	291	0.02	LN
Unit content of water	w	kg	189	0.01	LN
Unit content of sand	ρ	kg	853	0.015	LN
Unit content of gravel 4-8 mm size	$g_{4\text{-}8}$	kg	382	0.015	LN
Unit content of gravel 8-16	$g_{8\text{-}16}$	kg	630	0.015	LN
Mass density of cement	ρ_c	kg/m^3	3100	0.015	N
Mass density of sand	ρ_s	kg/m^3	2590	0.02	N
Mass density of gravel 4-8	$\rho_{g4\text{-}8}$	kg/m^3	2540	0.02	N
Mass density of gravel 8-16	$\rho_{g8\text{-}16}$	kg/m^3	2660	0.02	N
Concrete cover	c_c	mm	20.2	0.2	N
Diameter of steel	D_s	mm	22	0.05	LN
Height of cross-section	H	m	1.25	0.02	N
Top flange depth	H_1	m	0.15	0.02	N
Top flange width	B_1	m	0.3	0.02	N
Web thickness	B_2	m	0.15	0.02	N
Strength of concrete in compression	f_c	MPa	54.1	0.15	LN
Strength of concrete in tension	R_{bt}	MPa	3.1	0.25	LN
Strength of steel	R_s	MPa	410	0.05	LN
Modulus of elasticity for steel	E_s	GPa	210	0.03	N
Modulus of elasticity for concrete	E_b	GPa	23	0.05	N
Type of cement	C	-	1.4	0.247	U <0.8; 2>
Influence of humidity	k	-	0.65	0.311	U <0.3; 1>
Influence of CO$_2$ content	d	-	1.5	0.192	U <1; 2>
Current density	i_{corr}	μA/cm^2	1.5	0.33	U <1, 2>

Remarks: N...Normal, LN...Lognormal and U...Uniform on interval <X_{lower} ; X_{upper}>

In Fig. 5 a) homogenous corrosion is treated and updated values of carbonation depth (smaller values) decreased the failure probability to the level of the failure probability for zero time. For pitting corrosion, this decrease to zero time level works approximately up to 40 years, then there is an increase of the failure probability (but of lower values), see Fig. 5 b). It has to be realized that we start from time zero where a failure probability exists ($\approx 10^{-5}$) and this failure probability is the result of the random variability of geometrical and material characteristics only. Also the influence of two different models for the carbonation process was studied. The curves in Fig. 6 indicate only negligible effects in this particular beam. A more serious degradation effect is the pitting corrosion. Using the same approach also the influence of a type of corrosion on the probability of failure was studied - as it can be observed in Fig. 7. The results are only indicative as the pitting process is strongly uncertain and is often started due to the chlorides attack.

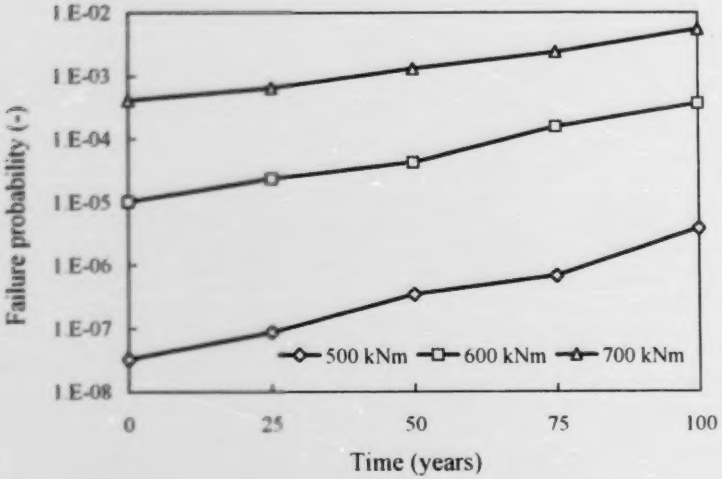

Fig. 3: Failure probability vs. time for different levels of bending moments (carbonation model by Papadakis and homogenous corrosion).

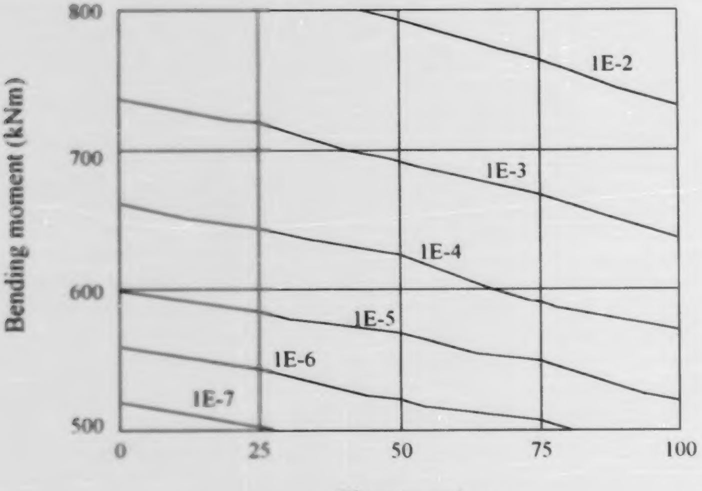

Fig. 4: Isoprobabilities (Papadakis, homogenous corrosion).

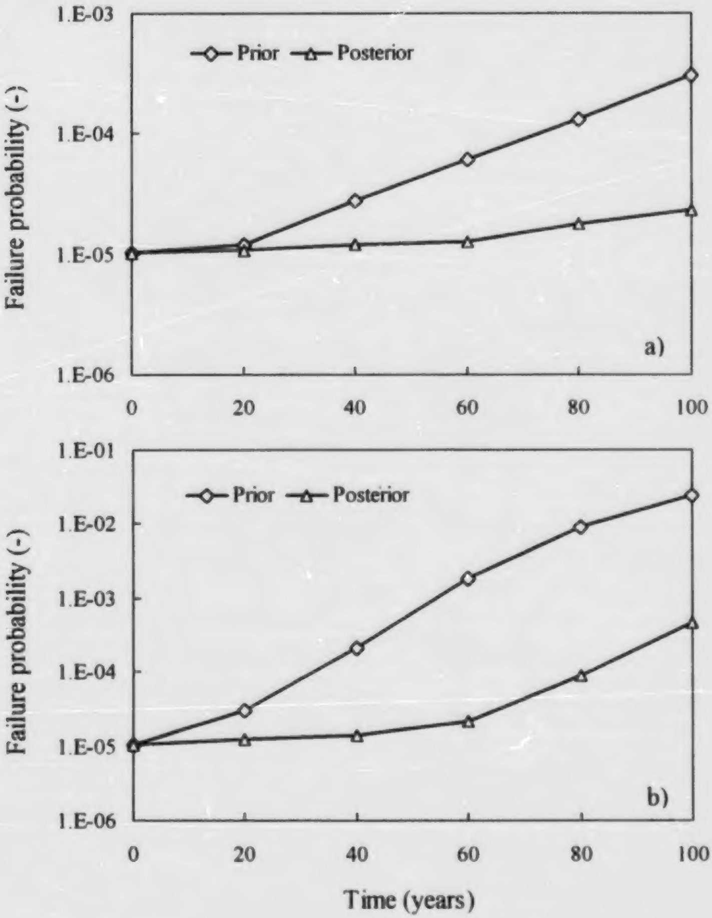

Fig. 5: Bayesian updating of p_f (carbonation model by Papadakis and M= 600 kNm): a) homogenous corrosion; b) pitting corrosion.

6 Conclusions

- The influence of the degradation process on reliability is modelled and shown in a quantitative probabilistic way - failure probability vs. time.
- The approach described above may conveniently serve for the assessment of the residual life of a RC structure taking into account the degradation due to carbonation of concrete and corrosion of steel reinforcement.

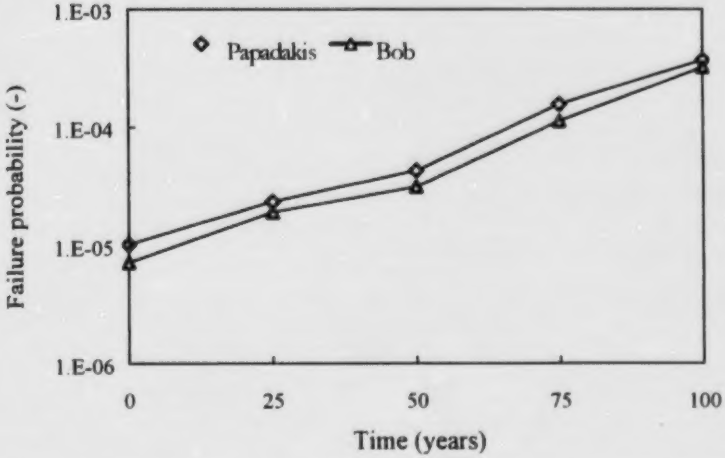

Fig. 6: Influence of carbonation model (homogenous corrosion and *M*= 600 kNm).

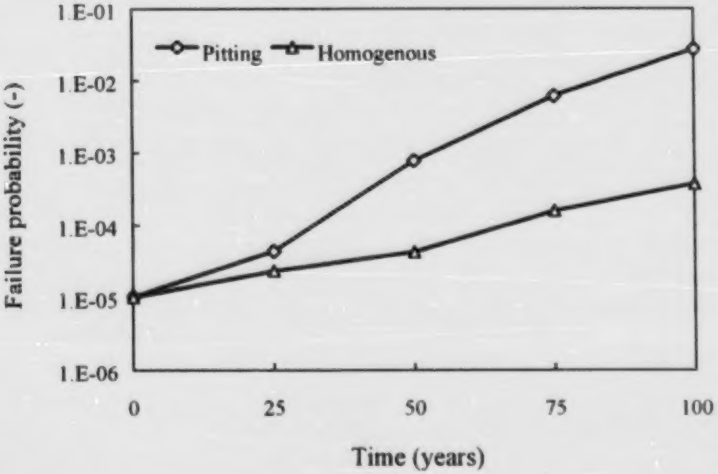

Fig. 7: Influence of type of corrosion - homogenous (α = 2) and pitting (α = 6) (carbonation model by Papadakis and *M*=600 kNm).

- Utilizing the Bayesian updating procedure and site-oriented measurements the predictions of degradation processes and the evaluation of their impact on the reliability measure are more effective. The local conditions are accounted for.
- The approach is rather general and different models and other degradation processes may be incorporated.

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8 References

Andrade, C., Sarria, J. and Alonso, C. (1996) Corrosion Rate Field Monitoring of Post - Tensioned Tendons in Contact with Chlorides. *Durability of Building Materials and Components 7 (Volume Two)*, London, pp. 959-967.

Bob, C. (1996) Probabilistic Assessment of Reinforcement Corrosion in Existing Structures. *Concrete Repair, Rehabilitation and Protection*, London, pp. 17-28.

Boer, A. and Veen, C. (1998) Simple Way of Finding Residual Strength of Actual Structures after Influences of Deterioration. *Diana World*, issue No.1, pp. 8-11.

Bourgund, U. and Bucher, C.G. (1986) *Importance Sampling Procedure Using Design Points - A User's Manual*. Internal Working Report, Institute of Engineering Mechanics, University of Innsbruck, Austria.

Keršner, Z., Teplý, B. and Novák, D. (1996) Uncertainty in Service Life Prediction Based on Carbonation of Concrete. *Durability of Building Materials and Components 7 (Volume One)*, London, pp. 13-20.

Lee, H.S., Tomosawa F. and Noguchi, T. (1996) Effects of Rebar Corrosion on the Structural Performance of Singly Reinforced Beams. *Durability of Building Materials and Components 7 (Volume One)*, London, pp. 571-580.

Lee, H.S., Noguchi, T. and Tomosawa F. (1998) FEM Anal. for Struct. Performance of Deteriorated RC Struct. Due to Corrosion. *CONSEC*, Tromso, pp. 327-336.

Matoušek, M. (1977) *Effects of some Environmental Factors on Structures*. Ph.D. thesis, Technical University of Brno, Czech Republic (in Czech).

Novák, D. Keršner, Z. and Teplý, B. (1996) Prediction of Structure Deterioration Based on the Bayesian Updating. *Proc. of the 4^{th} int. symp. on natural draught cooling towers*, Kaiserslautern, pp. 417-421.

Papadakis, V.G., Fardis, M.N. and Vayennas, C.G. (1992) Effect of Composition, Environmental Factors and Cement-lime Mortar Coating on Concrete Carbonation. *Materials and Structures*, Vol. 25, No. 149, pp. 293-304.

Rodriguez, J., Ortega, L.M., Casal, J. and Diez, J.M. (1996) Corrosion of Reinforcement and Service Life of Concrete Structures. *Durability of Building Materials and Components 7 (Volume One)*, London, pp. 117-126.

Stewart, M.G. and Rosowsky, D.V. (1998) *Structural and Serviceability Reliabilities for Chloride Diffusion, Cracking, Spalling and Corrosion of Concrete Bridges*, Research Report No. 162.02.1998, Department of Civil, Surveying and Environmental Engineering, The University of Newcastle, Australia.

PART NINE

GENERIC METHODS IN SERVICE LIFE PREDICTION

127 CRITICAL LOSS OF PERFORMANCE — WHAT FAILS BEFORE DURABILITY

A.M. Aikivuori

VTT Building Technology, Oulu, Finland

Abstract

Empirical research has been carried out to find out the actual reasons for initiation of repair projects on buildings. This research has shown that the owners of the buildings actually experienced the user requirements predominantly outside of the range of durability failures. Only 17 % of the repair projects were initiated primarily because of deterioration. The critical loss of performance seems to primarily be in the range of subjective perceiving of the building. Very little technical or economical rationality can be seen in the actual decisions made on building refurbishment. In most cases the limiting factor for service life is not durability.

Life cycle economics of the buildings are (according to the empirical findings) covered by technically/economically irrational basis. Decision-makers pay little attention to the condition and remaining potential for service life of the building components. They pay very limited attention to economical expectations. Optimization of economical factors of the buildings is the primary goal in less than 10 % of repair projects.

If service life is seen as the actual time in service of the building components, the basis of service life prediction models should not be based on durability or economics of the building components only. Durability is of course the limiting factor for service life in the sense that service life can not exceed the limitations set by durability, but in fact the actual service life seldom reaches the full potential life time of the components limited by durability. The forecasting of the timing of the refurbishment projects should not rely on the durability-based concepts only. Asset and maintenance management should pay more attention to the more critical perceived quality of the buildings.

Keywords: failures, technical rationality, economical rationality, optimization, actual service life, forecasting, service life prediction models

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1 Introduction

In this paper refurbishment is seen as an option at the end of the service life of a building. The end of service life occurs when the building fails to perform as required. This failure can be a result of three causes: 1) failure resulting from change(s) in performance (deterioration based failure); 2) failure resulting from change(s) in requirement(s) (obsolescence based failure) or 3) failure resulting from change(s) in use.

If the failure is due to changes in performance of the building, it is possible to:

i. have a corrective refurbishment to restore performance to its former level, or
ii. change the use of the building to match lower performance requirements or
iii. discard the building.

If the refurbishment is due to changes in the requirements, it is possible to:

i. have refurbishment according to new requirements or
ii. change the functional use of building so that its performance will meet the requirements of the new use or
iii. discard the building.

If the failure is due to change in space use, it is possible to:

i. have a refurbishment to match the spaces with the changed use or
ii. find another use for the building as it is or
iii. discard the building (perhaps temporarily until it is put to a new use).

A refurbishment project may be initiated suddenly because profound damage has occurred, or its initiation may be planned in advance for a chosen time according to the expected rate of deterioration. The damage-based refurbishment is most obviously due to the properties of the building (especially durability features). The other route to refurbishment is via changes in the use and the user of building with the normal wear and tear of the building itself. The theory of economy considers rational the project that maximizes economic utility. One may endeavor to do this directly through attempting to increase profit, or indirectly through minimizing expenditures in order to add profit potential (cost-reduction expenditure). A refurbishment initiated without deterioration of building on the basis of subjective valuations only can be seen as a counterpart of failure in building. A marginal factor for initiation of refurbishment projects can be anticipated to be a change of circumstances (e.g. change of energy price).

According to this, the reasons for demand of various refurbishment projects can be classified as follows:

1 Failure in building: corrective refurbishment
2 Change in use: space-altering refurbishment
3 Optimization of economical factors: optimizing refurbishment
4 Subjective features: pleasure refurbishment
5 Change of circumstances: opportunity refurbishment

Durability is the main concern of service life studies. Durability related deterioration is often seen as the process leading to the end of service life of structures. Service life prediction methods are based on assessing the extent of deterioration in some specific measurable properties chosen to be indicators of degradation (Sjöström 1985). The aim of service life prediction is to estimate the performance over time as the function describing how the measured values of the chosen properties vary with time. All this is related to the durability of materials. Durability itself is a complex issue. Deterioration, of course, impairs the capability of a system to meet the given requirements. But the requirements are not entirely covered by the capability of a system to maintain its performance. In the following, items other than deterioration are discussed as a counterpart to it as the criteria for refurbishment, and their power to explain the refurbishment projects is viewed.

2 Obsolescence and deterioration

There are two considerations in the process leading to refurbishment. On the one hand there are the requirements (which can be seen as the demand) and on the other hand there is the performance of the object (the supply). But what are the requirements that actually launch the refurbishment project? Or in other words, how is the demand for refurbishment initiated?

The refurbishment project of a building may be initiated for various reasons. Flanagan et al. (1989) who includes the following items makes one summary of the reasons from the life cycle analysis point of view:

- physical deterioration
- economic obsolescence
- functional obsolescence
- technological obsolescence
- social obsolescence
- locational obsolescence
- legal obsolescence
- aesthetic and visual obsolescence (fashion/image obsolescence)
- environmental obsolescence

Equivalent lists with to some extent different points of view have been written in a number of publications.

The level and to some extent also the character of requirements change over time. In general the requirements both rise and increase (for example the case of thermal insulation requirements in structures clearly shows this kind of development). Simultaneous to the rise in requirements is the decline in performance due to the deterioration of structures. The process of becoming discontent with the object accelerates. And even if a building could maintain its original properties, the refurbishment will finally be inevitable only because of the change in requirements due to the comparison between the existing situation and the new substitution available; in other words, there will be demand for obsolescence based refurbishment. Pullar-Strecker (1990) illuminates dearly the origin of the demand for refurbishment in his definition of building defects: "Building defects will always be with us because there will always be aspects of building performance that are less good than the best, or even average, or less good than we feel we have a right to expect." Thus the demand is derived from the comparison of the existing building performance with either standard performance criteria or personal expectations. It can be concluded that the defects are relative, not absolute, and therefore difficult to measure objectively. The refurbishment due to obsolescence is not technically rational since it does not optimize the use of structures until the end of their durability. Since corrective refurbishment is defined as the project aimed at deteriorated structures (an unacceptable physical condition), obsolescence based projects do not belong to this category. On the other hand, quoting Taylor (1980), deterioration has no part in the accumulated inferiority caused by obsolescence. Thus deterioration and obsolescence are two separable issues. Both Taylor (1980) and Flanagan & al. (1989) have categorized the causes of replacement into these two classes: deterioration and obsolescence.

3 Economic life - service life

The concept of economic life is defined by Taylor (1980) as the period of time that will elapse before the equipment - either present or proposed - is displaced from the intended service by more economically viable equipment. Or, restated, economic life is the period over which the equipment will continue to have the lowest annual cost compared to any contender for the service. Further, Taylor states that the concept of economic life is therefore inseparable from consideration of replacement. Replacement or displacement is the result of a successful attempt by superior equipment to terminate the service period of its predecessor.

In the technical-economical sense, maximization of utility means using an item until its failure so that the annual costs will be at their lowest. A normative definition for technical rationality can be expressed as: use as long as the item endures, maintaining its performance. Accordingly, a technically rational refurbishment project is initiated at the failure of the object. This area is handled in service life theory, which seeks to explain the relationship between deterioration and failure. According to service life theory, a technically rational refurbishment project will be initiated when service life meets its limiting lifetime.

4 Definitions and some aspects of service life studies

Service life is defined (by ASTM, 1990) as the period of time after installation during which all properties meet or exceed the minimum acceptable values when routinely maintained. Thus the object fails when property being assessed falls below the minimum acceptable value. In the following text, service life refers to the time of usage before failure of a building material, component or preferably the failure of a well-described system of building materials and components in an environment. Failure can be defined as the state of being where the object fails to meet the given performance requirements.

The shortest definition of performance requirement is given by ISO: performance requirement is the user requirement expressed in terms of the performance of a product. Performance is behavior (of a product) related to use (ISO 6241, 1984).

Performance criterion has been defined in slightly different ways. ASTM (1990) defines performance requirement as a qualitative statement of the performance required from a building component or material. Accordingly performance criterion is a quantitative statement of a level of performance for a selected performance characteristic of a component or material needed to ensure compliance with a performance requirement. RILEM (Technical Committee's TC 31-PCM Summary of Work, 1987) defines performance criterion as the limiting value or value range which satisfies the requirement for a given lifetime. (Here is the major difference between ASTM and RILEM definitions: ASTM determines the performance without any time concepts, whereas RILEM determines performance for a specific period of time.) Furthermore, for a defined requirement, the performance of a material is its response to the action of extrinsic factors. The nature of the response is due to the intrinsic properties of the material.

In other words these performance criteria are given as the minimum or maximum values of the properties vital to usability of the object. These kinds of definitions don't state either the properties or their acceptable values to be measured. Both properties and their limit levels are open to decision making. Thus it is obvious that, since the requirements are given according to standards with more or less variation, the definition of failure includes some non-absolute evaluation. Even the standards, which are given in legislation, vary in the course of time. This shows clearly that no estimate of service life can be absolute or final because it is a result of non-absolute and changing factors.

While trying to gain a service life estimate for a system, the first step has to be the identification of the requirements, which the system has to meet. After receiving information on the requirements, the performance criteria can be formulated. This is the phase where requirements are translated into levels of properties desired or demanded from the system. The use of the values of physical and chemical properties is essential for the practical task to gain a service life estimate, since the only things that can be physically measured in the system are its properties or the changes in the properties. In terms of the performance of an existing material system, we talk about the properties of the system. In terms of performance criteria, we talk about the requirements; these two modes have to be consistent.

The need of transforming the requirements into demanded levels of physical and chemical properties implies that the methodologies to study service life are indirect. This can be a source of inaccuracy for service life estimates since it is to some extend a matter of choice of how to describe the requirements in the terms of properties. Nevertheless, as soon as the properties have been identified, the measurements can be carried out more objectively.

The definition of service life as a period of use before failure implies that the service life studies actually seek to determine the length of time until the occurrence of failure and concentrate rather on the deterioration rate than on the remainder of beneficial properties of the system. That is indeed the case. The challenge in service life prediction is to determine or predict how long time a system will be able to maintain its desired or demanded properties above a level of acceptance. Usually systems are designed to meet some given requirements at the end of their construction or at the beginning of their usage. Hardly ever does improvement occur in the way in which the system is able to meet these requirements during its use. On the contrary, the usefulness of the system is usually susceptible to degradation both because the standards of the requirements tend to rise and, at the same time, the capability of the system to meet even the original requirements deteriorates.

A system meets its requirements according to its properties. Failure is usually expressed as the exceeding of a maximum or falling below a minimum required level of the properties which have been chosen to be characteristic of and demanded for proper performance of the system. The information on the in-use-behavior of the system is derived from the measurements of the changes in these properties. Time can be connected to the measurements as the rate of changes or, since the changes usually are to the worse direction, the rate of deterioration.

5 Empirical findings

The method chosen for empirical study was to identify the prime cause of refurbishment by means of a written questionnaire sent to those who have made a decision to initiate a refurbishment project. The method of asking decision makers directly was chosen because it could be directed at the origin of the information: only those who actually made the refurbishment project decisions are qualified to name their specific reasons. The written questionnaire was formulated in such a way as to gather facts about the background of the refurbishment and to pose the question of the motive. Since the motive question is crucial, it was asked both as a multiple-choice question and as a freely expressible question.

In this case, the empirical study was aimed at the whole population of private sector refurbishment projects permitted during the period from the 1st of July 1990 to 30th of June 1991 in the city of Oulu (Aikivuori 1994). There were the total of 270 cases including 140 private individuals, 91 housing corporations, 10 real estate corporations, 22 companies and 7 communities. The total answering percentage was 67 % and the total loss 33 %. Considering the reliability of the results, this answering percentage is dependable enough.

The empirical findings as to the prime causes for the refurbishment projects of this study are:

(1) failure due to deterioration: 25 cases, 17 %
(2)change in use: 38 cases, 26 %
(3)optimization of economic factors: 14 cases, 9 %
(4)subjective features of the decision maker: 65 cases, 44 %
(5)change of circumstances: 6 cases, 4 %

This distribution is valid only locally and only at the time of the study, each decision environment gives its own results.

Service life prediction based on the deterioration rate of structures covers 17 % of the observed cases (only 17 % of the refurbishment was initiated primarily because of deterioration). Based on the discussion above in the chapter 4, it can be said that deterioration based service life prediction methods give the maximum expected service life rather than a period to use in life cycle costing analyses.

The average periods of use before refurbishment in the categories are:

(1) failure due to deterioration: 28.7 years
(2)change in use: 26.8 years
(3)optimization of economic factors: 17.7 years
(4)subjective features of decision maker: 17.2 years
(5)change of circumstances: 24.7 years

The overall average time in use before refurbishment of all the cases is 22.0 years.

It can be seen here that the deterioration-based refurbishment occurs in average clearly later than its subjectively based counterparts. Changes in use or circumstances confirm the widely used period (in life cycle analyses) of 25 years, and the overall average 22.0 years is also reasonably close to it. According to these empirical findings, obsolescence based refurbishment occurs much earlier (in average, after 20.6 years in use) than deterioration based refurbishment (occurring in average after 28.7 years in use).

6 Discussion and results

These results indicate that the service life of a building component is more frequently determined by factors other than durability failure. Refurbishment projects causing the replacement of a building component occur prior to discarding of the component on technically or economically rational basis. It seems that decision-makers don't conduct in technically economically rational manner. The rationality in the majority of refurbishment decisions is very subjective in nature.

The study of the empirical material demonstrated that human behavior related to decision making in refurbishment projects is not a clear case of seeking a technical-economical optimum. Even if human behavior is not rational in terms of technology or economy, it is intended to be rational when choosing the best among alternatives. Obviously the greatest expected utility has been subjective in nature. The aspiration to seek pleasure seems to be a strong motivator for refurbishment. It also indicates that the built environment does not offer such pleasure, as people feel is their right to expect from it. Construction companies should benefit from the innovation and development of comfort providing items (such as the glazing of balconies in some environments) offered to the market.

The importance of failures in durability as a prime cause of refurbishment is surprisingly moderate (17 % of the observed cases). This low proportion is probably an indication of maintenance work being effective to repair a large portion of deterioration based failures.

If the occurrence of a refurbishment project in the life cycle of a building is seen as a forecasting problem, it is obvious from the empirical findings that the long established period of 25 years is a good estimate. On the whole, refurbishment projects take place on average when the building is 25 years old, and occurs again after another period of 25 years. The observed deviations from the average are however large, thus it is rather difficult and inaccurate to try to estimate the exact time at which a specific building will be encountering its refurbishment. For design periods and cost estimation purposes 25 years with its multiples is a solid basis, even regardless of whether the project is due to deterioration or obsolescence.

Service life prediction based on the deterioration rate of structure covers 17 % of the observed cases. Therefore it can be said that deterioration based service life prediction methods give the maximum expected service life rather than a period to use in life cycle costing analyses.

It is also apparent that the empirical information about both the reasons and timing of refurbishment projects is valid for statistical considerations only whereas it might be difficult to find single cases that exactly obey the averages of the findings.

Obsolescence is overwhelmingly a more important basis for refurbishment than deterioration (whereas deterioration may be the dominant cause for maintenance activities; this also implies that most durability failure costs appear as maintenance costs and not as refurbishment costs). To maximize the utilization of structures, factors causing early obsolescence should be identified and improved.

7 References

Aikivuori, A. (1994) *Classification of Demand for Refurbishment Projects.* Acta Universitatis Ouluensis. Series C Technica 77. University of Oulu Printing Center. 71 p.

ASTM (1990) Standard Practice for Developing Accelerated Tests to Aid Prediction of the Service Life of Building Components and Materials. s. 514 - 519. In: 1990 *Annual Book of ASTM Standards.* Section 4 Construction. Volume 04.07 Building Seals and Sealants; Fire Standards; Building Constructions. Easton, MD, USA. American Society for Testing and Materials. 1078 p.

Flanagan, R., Norman, G., Meadows, J. & Robinson G. (1989) *Life Cycle Costing Theory and Practice*. Oxford. BSP Professional Books. 181 p.

ISO 6241-1984 (E). (1984) *Performance standards in building - Principles for their preparation and factors to be considered.* International organization for standardization. 10 p.

Pullar-Strecer, P. (1990) Building defects: Are They Avoidable? In: Quah, L. K. (ed.) Building Maintenance and Modernisation Worldwide. Volume one. Singapore. Longman. pp. xlvii - lii.

RILEM Committee 31-PCM. (1986) *Performance criteria for building materials, Final Report.* Espoo. Technical Research Centre of Finland. Research Notes 644.

Sjöström, C. (1985) Overview of Methodologies for Prediction of Service Life. In: Masters, L. W. (ed.) 1985. *Problems in service life prediction of building materials and components.* Dortrecht, The Netherlands. Martinus Nijhoff Publishers. pp. 3 - 20.

Taylor, G., A. (1980) *Managerial and Engineering Economy. Economic Decision-making.* 3. ed. New York. D. Van Nostrand Company. 538 p.

128 DURABILITY OF SOIL CONTAMINATED BY OIL

Y.J. AL Shakarchi and H. A. Rasool
University of Baghdad, Iraq

Abstract

The durability of soil as a foundation material contaminated by oil is one of the problems encountering soil engineers. The contamination of the soil is due to the leaks from the oil tanks. In order to study the effects of contamination on the durability of soil as a foundation material, a number of controlled laboratory experiments were carried out using disturbed and undisturbed samples before and after contamination. The soil tested was clayey silt. Disturbed samples were exposed to varying concentration of oil. The oil was added at four concentrations 5, 10, 15 and 20%. The undisturbed samples were exposed to oil for different periods of immersing. The physical properties as well as the mechanical properties were examined. Triaxial compression machine and direct shear apparatus were used to study the behaviour of compacted specimens under the effect of oil contamination. Standard Oedometer was used to examine the compressibility of undisturbed specimens and compacted specimens.

Keywords: Contamination, durability, oil, shear strength

1 Introduction

Nearly every gallon of oil used as a fuel, is stored in underground storage tanks. Some of these tanks or associated pipework may be leaking (3). One of the problems of leaking oil storage tanks that have encountered soil engineers is the durability of soil as a foundation material contaminated by oil. The problems of leaking has been mentioned in the references (1,4).

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The work reported here covers two separate, but closely related parts. The first part deals with the physical properties of the contaminated soil. The second part describes the mechanical properties such as shear strength and compressibility. The conclusions are based upon the combined assessment of the results of these two parts.

2 Experimental work

Block samples were obtained from Dora site near Baghdad city. A narrow pit was excavated to a depth of 2 meters. Disturbed samples as well as undisturbed samples were obtained. The undisturbed samples were obtained from irregular block excavated from the pit. The blocks were trimmed carefully to get undisturbed samples for shear and compressibility tests.

Kerosene was the oil product chosen in the study because, it is widely used and has low viscosity which permits easy penetration in the soil. As the oil penetrated the soil, its chemical nature appeared to be altered by natural chromatographic processes.

In order to study the effects of contamination on the durability of soil as a foundation material, a number of controlled laboratory experiments were carried out using disturbed and undisturbed samples before and after contamination. Compacted specimens were also tested.

2.1 Physical properties

Disturbed samples were exposed to varying concentration of oil. The oil was added at four concentrations 5, 10, 15, and 20%. The soil used was air dried at 25°C.

The grain size distribution, specific gravity and Atterberg limits were tested before and after contamination. Standard compaction test was also performed. All soil samples were tested in accordance to ASTM standards (2). The test results are shown in table 1. It is clear that contamination of soil by oil affects the physical properties of the soil. Gradation as well as Atterberg limits are changed. Liquid limit as well as plastic limit are increased with the increase of oil mixed, while plasticity index remains nearly constant. Specific gravity decreases with the increase in oil concentration. Results of standard compaction tests indicate that optimum moisture content increases with the increase in contamination and the maximum dry density decreases. Certainly, the change in the physical properties due to contamination of soil with oil will affect the other properties such as mechanical properties of the material.

2.2 Mechanical properties

Direct shear apparatus as well as triaxial compression machine were used to study the effect of oil contamination on shear strength parameters of natural soil. The direct shear tests were conducted on compacted specimens, 60mm x 60mm x 20mm, at optimum moisture content with varying percentages of oil mixed with natural soil. The results of standard compaction tests are given in table 1. Results of unconsolidated undrained direct shear tests are summarized in table 2.

It is clear that contaminating natural soil with oil products only slight increase in angle of internal friction and a significant decrease in cohesion. The combined effect of two is a reduction in shear strength of soil contaminated with oil.

Unconsolidated undrained triaxial tests were also performed on compacted contaminated specimens. The dimensions of the specimens were 38mm.dia. x 76mm long. It should be noted here that difficulties like fissuring were encountered during sampling operation. As a result of that, only two concentrations of oil 5% and 10% were used in mixing with natural soil during sampling. The results are shown in table 2. It could be noticed that there are close agreements between results of direct shear test and triaxial tests. The reduction in cohesion may be attributed to the fact that clay does not become plastic when mixed with a liquid of non-polarizing molecules like kerosene.

Table 1: Physical properties of soil

	Percent of oil mixed				
Gradation (%)	0	5	10	15	20
Silt	62	66	73	76	78
Clay	38	34	27	24	22
Specific gravity	2.72	2.70	2.69	2.66	2.57
Atterberg Limits					
Liquid limit	37	41	43	45	47
Plastic limit	23	27	28	30	32
Plasticity Index	14	14	15	15	15
Standard Compaction					
Optimum Moisture content (%)	20	20.7	23.5	25.5	30.3
Maximum dry density g/cm^2	1.675	1.664	1.598	1.522	1.461

Table 2: The results of shear tests

	Percent of oil mixed				
From Table 1	0	5	10	15	20
Optimum moisture content (%)	20	20.7	23.5	25.5	30.3
Maximum dry density (g/cm^3)	1.675	1.664	1.598	1.522	1.461
Direct shear test					
Friction angle (deg.)	2	4.2	5	5.5	19
Cohesion (kN/m^2)	105	52	26	10	4
Triaxial shear test					
Friction angle (deg.)	2	4.8	6.5	–	–
Cohesion (kN/m^2)	100	50	25	–	–

2.3 Compressibility

Two groups of undisturbed specimens, 75mm dia. x 19mm high, were tested using standard oedometer to study the consolidation characteristics of the soil. The first group deals with specimens soaked in water prior to testing while the second one deals with specimens soaked in oil. It should be mentioned that the two specimens used were obtained from one location and have the same initial properties. The consolidation tests were performed after immersing the first one in water and the second one in oil.

Table 3: Results of consolidation

	e	c_c	c_r	ε_v %	ω %
Undisturbed specimens					
Specimens soaked in water	1.466	0.36	0.060	20.6	22.5
Specimens soaked in oil	1.016	0.30	0.031	17.5	22.5
Compacted specimens					
Natural or 0% oil	0.914	0.19	0.080	10.5	20
5%	0.632	0.21	0.014	13.7	20.73
10%	0.717	0.13	0.002	15.7	23.5
15%	0.746	0.15	0.003	19.6	25.5
20%	0.754	0.14	0.019	20.2	30.3

e = void ratio c_c = compression index ε_v = volumetric strain
ω = moisture content c_r = swelling index

Table 3 shows the results of test for the two groups. It could be noticed that initial void ratio after immersing the soil in oil is less than void ratio of the specimen immersed in water. This behaviour may be attributed to the fact that swelling potential in specimens soaked in water is more than that in specimens soaked in oil.

Similar trend could be observed with respect to compression index, swelling index and volumetric strain.

Compacted contaminated specimens were also tested. The results of consolidation are given in table 3. The tests were conducted on specimens compacted at optimum moisture content. Again one could notice that all contaminated specimens with oil have smaller values of compression index, swelling index and volumetric strain than that of natural specimens. At the same time the effect of increasing oil content have no clear trend on the values of compression index and swelling index. Nevertheless, the volumetric strain increases with the increase in oil content.

3 Conclusion

The oil content in soil will increase the possibility of settlement in the foundations of the constructed facilities because of the significant change in the properties of soil. Based on test results, the following conclusions could be drawn:

1. A significant increase in liquid limit and a slight increase in plastic limit with the increase of oil content.

2.The optimum moisture content increases and the maximum dry density decreases with the increase in oil content.

3. A significant reduction in cohesion and a slight increase in friction with the increase in oil content.

4 References

Al Daibaki, A.S. (1983) *Effect of some petroleum derivatives on some soil hydrophysical properties, M.Sc. Thesis*, Baghdad University.

ASTM (1983) *Annual Book of Standards, Philadelphia*

"Confluence," (1985) *Environmental Science and Technology*, Vol. 19, No. 5

Hoag, G.E., and Marley, M.C. (1986) Gasoline residual saturation in unsaturated uniform aquifier materials. *Journal of Environmental Engineering*, Vol. 112, No. 3. June.

129 DURABILITY ASPECTS OF FIBRE REINFORCED COMPOSITES

S.J. Bhide and M.M. Zurale

Maharashtra Institute of Technology, Pune, India

Abstract

FRP have been used in many civil engineering applications, such as short and long span bridges, buildings, off-shore structures, retaining walls etc. FRP will certainly play a major roll in the construction, rehabilitation and repair of civil engineering structures in the 21st century. The high strength-to-weight ratio and non-corrosive characteristics of these materials could be utilised to build innovative structures that can not be built, using the current conventional materials. FRP could be used to build light, yet more durable and economical structures. Designers should be encouraged to utilise these materials based on its own unique characteristics rather than a replacement for current conventional materials. Eventhough fibre reinforced plastics (FRP) composites are being explored for many applications, including transport systems, offshore structures, marine structures, the durability issue of FRP composites has remained an open question with a composite structural designers.

Keywords: reinforced composites, fibres, resin, durability, failure, laminates

1 Introduction

The problems of building and housing are particularly acute in the developing world and the struggle to provide shelter to the teeming millions at affordable costs, has surfaced as the issue of highest importance the world over. The search for new and alternative building materials will have to be intensified, because, conventional materials are fast depleting. The tremendous variability in the geo-climatic conditions, the life style of the people, the affordability levels and the nature and quantum of available building materials, underline the urgent need for the identification of right materials, with their durability. Appropriate

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technology packages, with respect to material characteristics, based on climate of the region and felt needs of the users, are also equally important.

There has been a rapid growth in the use of fibre reinforced plastics (FRP) materials in the civil engineering applications, during last few years. However, the full potential use of FRP technology has not been fully realised for civil and structural engineering applications. FRP is currently produced in the form of laminates, doors, windows, partition panels, structural sections, reinforcing bars, grids, and pre-stressing tendons. FRP materials should not be treated as a direct substitution for conventional materials. However, it should be utilised according to its own fundamental characteristics. The advantages of FRP should provide the designers, the opportunity to design structural systems, that could not be built, using conventional materials.

One of the major problems, that readily reduces the life of concrete structures, is the corrosion of steel reinforcement. This problem is more serious in the cold climate countries, where de-icing salts accelerate the deterioration. Since, FRP are non metallic materials, they have major advantages over steel. Use of these materials could significantly increase the life time of structures, minimising the maintenance requirements and consequently off-set the current high costs of FRP products. The wide spread deterioration of infra-structure due to age, environmental factors and steadily increasing high way traffic patterns, (load and frequency) has created a critical need for cost effective and durable materials and technologies, for use in rehabilitation and retrofit. Fibre-reinforced polymer composites present an attractive alternative, for use in the retrofit and new construction. The high specific strength and stiffness, light weight, corrosion resistance, and enhanced fatigue resistance, combined with capability, to easily mould complex shapes, makes the composite attractive, when compared to steel and concrete.

Along with other properties like workability, durability is a very important property, especially, in building materials and various components. It is the property of FRP in particular, by virtue of which, it is capable of resisting its disintegration and decay, which may be caused due to:

i) use of unsound resin
ii) use of less durable aggregate
iii) entry of harmful gases through the pores and voids, present in FRP components and structures.
iv) freezing and thawing of water, sucked through the cracks or crevices by capillary action.
v) expansions and contractions resulting from temperature changes and alternative
vi) drying and wetting.

Fibre reinforced polymer composites are being explored for many applications, including transportation systems, off shore structures, marine structures, and infrastructure. However, durability issue of fibre reinforced polymer composites, has remained an open question to the composite structural designers. When a specific application is contemplated, it is essential for the design experts to know, not only the answer to the questions regarding strength and stiffness; but also the question of the life of the material under the conditions anticipated. Many factors play roles in reducing the life of the component. In most cases, it is difficult to define or estimate the life of the component. Characteristics of most of the materials, change by interaction with their surroundings, over a period of time. Oxidation of steel, leads to corrosion and it is a well known fact that, water accelerates the process. Concrete degrades in harsh environment. Therefore it is advisable, that before using FRP composites, we must know the possible changes to happen to FRP composites, under the given service conditions. A knowledge of these changes before hand, can allow us to accommodate them in the design, and to accommodate these changes in the design, the data on rate of change and effect of these changes on the behaviour, must be very well obtained and evaluated. Unfortunately, reliable data on systematic basis on durability, is not easily accessible.

In consideration of life cycle (durability), it is important to define, the limit of life. Some materials may not fail or isolate under certain service condition; but can continue to deform or deflect beyond an acceptable limit. Thus effective life ceases, when pre-determined de-formation or deflection limit is exceeded.

2 Mechanism of degradation

2.1 General

The mechanism of degradation is an important factor, to be considered in the discussion of durability. The mechanism which controls the durability of composites, is well known. This includes:

i) chemical or physical changes or micro cracks of polymer matrix.
ii) loss of adhesion or debonding at the fibre/matrix interface.
iii) reduction of fibre strength and modulus.

All above mentioned processes contribute to the growth of flaws or cracks when failure occurs. For unidirectional composites, the compressive failure mechanisms include:

i) micro buckling of fibres
ii) kink banding
iii) matrix yielding and
iv) shear through fibre and matrix .

The tensile failure modes are:

i) brittle failure of the matrix
ii) matrix brittle failure with fibre pull out
iii) interface matrix shear failure and constituent debonding.

In case of multi directionally reinforced composites, the initial failure takes the form of cracking of constituent plies, i.e. cracks appear parallel to fibre, long before the final failure of the laminates. The initial failure is very often referred as first ply failure.

2.2 Environment

Environment plays a very crucial role in changing the properties of polymer matrix composites. The environment, both ambient and loading, can affect the durability of the composites. In ambient environment, both matrix and fibres may be affected by moisture, temperature, ultraviolet radiation, ozone and the presence of degrading chemicals like salts or alkalies. Repeated exposure to very low and high temperatures (freeze thaw cycling), too, may undergo some changes. There are various factors, which affect the durability of FRP composites which include:

i) Aging of polymer matrix.
ii) influence of moisture.
iii) low temperature effects.
iv) freeze thaw effects.
v) influence of ultra violet rays.
vi) creep behaviour.
vii) fatigue behaviour.
viii) fire hazards.

2.2.1 Aging of polymer matrix

Durability of composites, must be considered in the context of the role of polymer matrix, as it is most susceptible to changes, in response to environment. The role of matrix in the composites, is to transfer stresses between fibre, to provide a barrier against an adverse environment and to protect the surface of the fibres from mechanical abrasion. Matrix has a major influence on the interlaminar and in-plane shear load transfer. Therefore, if the matrix itself changes it's characteristics with time, the consequences need to be carefully studied. The environment, temperature and the humidity control, this change. This process is known as aging. The aging occurs, when polymer is cooled below it's glass transition temperature (Tg), the material does not achieve instantaneous thermodynamic equilibrium. As a result of physical aging, some polymers can become stiffer and brittle, influencing the matrix dominating properties, namely, the shear and transverse response. The effects on the fibre dominated properties, will be minimum. Thus in designing with composites, it is necessary to consider the fibre orientations relative to major loading axes.

2.2.2 Influence of moisture

Many polymer composites, when exposed to humid air or water environments, absorb moisture by diffusion and instantaneous surface absorption. In desert conditions, the converse would apply, as relative humidity is very low, moisture being given off. The rate of water uptake (release) and its equillibrium concentration, as well as it's influence on properties, could therefore, be of great concern.

2.2.3 Influence on matrix

Absorption of water by resin, may change the resin properties and glass transition temperature, through hydrogen bonding, of water molecules. The stiffness of the composite may drastically degrade, if the glass transition temperature is lower than the moisture absorption temperature. Moisture absorption is benefecial to composites. Absorption of water, causes swelling of the resin. The swelling of the resin matrix in composites, around the fibre, reduces the residual compressive strength at fibre/matrix interface, caused by the curing shrinkage. This results, in release of mechanical inter-locking stresses between, the fibre and the matrix. Voids present at the fibre/matrix interfaces or in the layer of the composites, can cause plastification of the resin. Water, entrapped in the voids, may cause blisters.

A systematic study has been carried out on kinetics of absorption as a consequence of prolonged immersion in water (Pritchard. and Speake 1987). The data for the pure resin and laminates are shown in Figure 1 and 2.

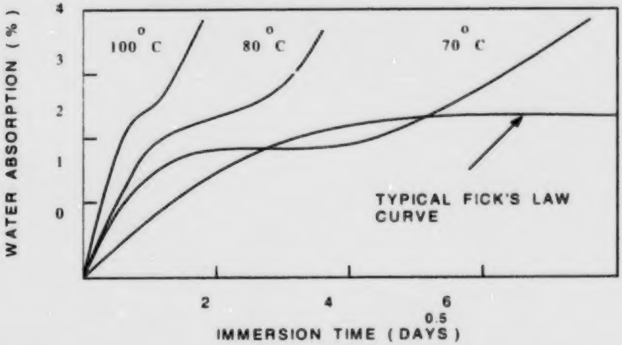

Fig. 1: Water absorption kinetics of pure iso-polyster resin

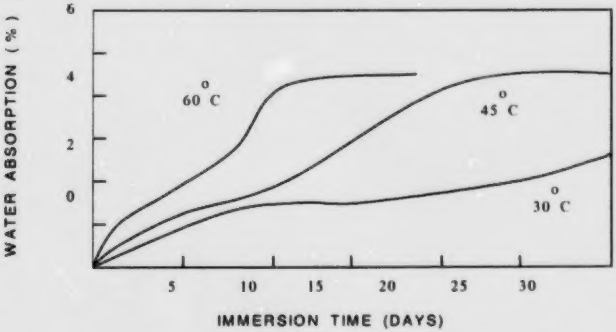

Fig. 2: Water absorption kinetics of glass reinforced iso-polyster resin

The water absorption uptake, of both pure resin and laminate, depends upon temperature. The higher the temperature, quicker is the plateau value. The results have been analysed in terms of Fick's Law and the activation energies for the water uptake and desorption of various materials, in the following table.

Table.1: Water uptake and water desorption of resins

Resin	Water uptake KJ/mole	Water desorption KJ/mole
Iso-polyester resin	55.4	187.6
UD glass/polyester	73.4	175.8
Polyester laminates	45 to 51	--
Epoxy resin	55.6	--
Glass/epoxy laminate	44 to 48	--

2.2.4 Influence of fibres

It is believed that, glass fibres can be damaged by prolonged exposure to water. The glass is silica, in which oxides of alkaline metals are dispersed. Alkaline metallic oxides, are hydroscopic and hydrolyzable. E & S glass contain only small amounts of alkaline metal oxides and hence are resistant to damage, by water. Presence of water, at the glass fibre interface, lowers its surface energy, which promotes the crack growth.

Moisture does not have any known degrading effects on carbon fibres. Aramid fibres can absorb considerable quantities of water, resulting in swelling.

Most of the fibres undergo sizing that ensures good bonding with matrix and protect them water absorption.

2.2.5 General behaviour

Water saturated composites, usually display increased ductility, due to softening of the matrix. This is advantageous in polymer composites and should not be ignored. These degradation changes are mostly reversible

2.2.6 Hygro thermal effects

Temperature plays a crucial role in the water absorption mechanism of composites. Temperature influences the distribution of water, the quantity and rate at which, it is absorbed. Damages, induced by the immersion in boiling water for few hours, produce debonding and cracking of same degree of damage as at 50°C over a period of 200 days.

2.2.7 Alkaline environment

It is essential to use, in this environment, alkali resistant glass, as alkaline solution reacts with glass fibres to form expansive silica-gels. This precaution is specially important for application of glass fibre reinforced composite material, as reinforcing bars in concrete. GRP bars are being increasingly considered to replace steel reinforcing bars, which are corroded by environment or de-icing road salt. During hydration period of concrete, highly alkaline ($pH > 12$) pore water solution is created. This highly alkaline solution can affect the glass fibre and reduce the durability of the bars. High alkali resistant glass (C-glass) can improve the durability of such cementeous application. Generally glass / polyester has resistance to most chemicals, except strong bases and strong oxidants (Mayer1985).

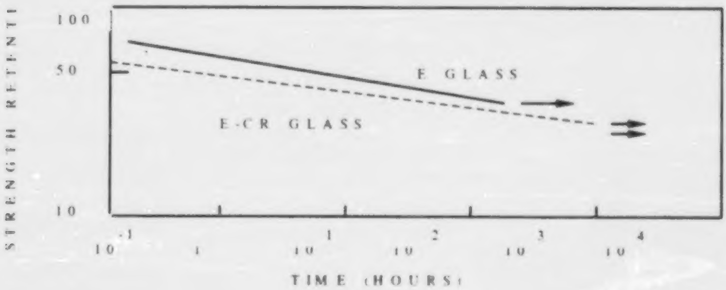

Fig. 3: Tensile stress corrosion of laminates in sodium hydroxide at 30°C

2.3 Low temperature effect

Extreme changes in temperature of composite materials result in several important effects. The coefficient of thermal expansion (CTE) of the matrix is usually greater in order of magnitude, than that of fibres. Decrease in temperature, will cause matrix to shrink. Contraction of the matrix is resisted by relatively stiff fibres, through fibre/matrix interface bonding. Energy absorption, before failure at low temperature, is higher, than energy absorption at room temperature.

2.4 Freeze-thaw effects

Unless a composite contains, a significant percentage of inter connected voids, which are filled with water, the freeze thaw effect on strength within a temperature range (+30°C to-20°C) is insignificant. Commercially available GRP composites, contain about 0.4% voids, which do not allow any frozen moisture to cause any serious damage. Under extreme low temperature conditions, the stresses can result in the formation of a micro cracks in the resin fibre inter-face. Chances of micro crack growth, under normal range of service temperature (+30°C to -20°C) are marginal.

2.5 Influence of ultra violet rays

The effect of ultraviolet rays on polymer composites is welknown. Long time exposure to sunlight hardens the matrix and is discoloured. The problem is generally overcome by incorporation of u-v resistant additives in the matrix. In thicker composites, the effect is minimum and structural properties are not much affected, as it is only a superficial phenomenon.

2.6 Creep behaviour

Creep is the increase in strain with time, at constant stress level. Creep occur, because of combination of elastic deformation and a viscous flow. In composites, the creep strain depends on the stress level and temperature. It is a function of both distribution of stress between the fibre and matrix. Highly cross linked thermosetting matrix, show lower creep rates than thermoplastic polymer composites.

Glass and aramid fibres over a long period of time exhibit failure by stress rupture. Carbon fibres are less prone to stress rupture.

2.7 Fatique behaviour

Fatigue properties of structural materials, are their response to cyclic loading. The composites are generally well known for their excellent fatigue behaviour. There are many aspects to consider, which include type of loading, load duration, load introduction, loading rate, temperature and finally, environment.

The fatigue is also influences by the fibre alignment. Combination of fabrics, which contain a mixture of fibre alignment, are often chosen to ensure, that, there are fibres in all directions with more fibres in a direction with higher loading.

The fatigue performance of E & S glass fibre reinforced composite, is poorer than that of carbon-aramid fibre composite.

2.8 Fire hazards

FRP composites used for any structural construction must comply with local construction code requirements, including fire resistive and other life safety specifications. At present, codes for structural composites in building are not extensive. Designers have to deal with fire hazard characteristics of the material. The materials are tested by the following test methods.

Uniform Building code (U.B.C.) standard 52-3 (ASTM - D 1929) forself ignition. Smoke density - (ASTM - E 84)

Specimen will be approved in accordance with UBC standard 524 (ASTM-E 84).

Finally the designer needs to decide, the necessary level of fire resistance and methods to attain and measure of fire safety. Fire tests, like non-combustibility, ignitability, flame spread, rate of heat release, smoke and toxic gas emission, flash over etc. are of prime importance to consider the durability factor.

3 Durability of materials under random exposure conditions

All materials deteriorate, when exposed to aggressive environments. This affects, significantly the strength and durability of structural materials and components. The damage to materials due to an aggressive environment can be in the form of cracking, accumulated permanent strain, or even decolouring. Any damage influences the life and functionality. It is of significant practical interest to assess analytically to comulative damage to a material, when it is exposed to random environmental conditions. While considering durability issue, it is better to know the damage phenomenon. It is dependent on a number of factors. Experimental studies indicate, that the damage rate can be approximated by an equation.

$$\frac{dD}{dt} = AC^{m}D^{n} \text{ where,}$$

t = time of damage,
C= Environmental factor that affects damage
A, m and n are constants.

The environmental factors that affect damage are temperature humidity and concentrations of a chemical in the environment. In real life situations, the variation of C with time is not a definite quantity. But it is a random process.

Durability of Glass Composites, under alkaline environment is really harsh. FRP bars with urethane modified bisphenol vinylester if used, show lowest vulnerability. Bars with isocyanurate vinyl-ester resins (IVE) exhibit, superior strength and stiffness in unconditioned salt and freeze thaw environment.

4 Repair of structures

A number column, slabs and girders have been repaired and strengthened with CFRP products, due to earthquake damage and structural needs to increase their capacity. Many products are currently available for this type of retrofitting. The products are often unidirectional and produced in the form of fibre tapes, fibre winding strands and fabrics. The materials are effective for both flexural and shear strengthening of structures.

A retrofit process of a structure, such as a chimney, begins with preparation of concrete surface, trowelling the surface with epoxy, followed by placement of the auto-adhesive tapes in the longitudinal direction and confining of the outer surface in the circumferential direction, by winding small diameter carbon cables, for which automatic winding machine is available.

4.1 Repair of bridges

Many highway bridges deteriorated due to continuous increase of the truck capacity, legally permitted on highways. The repair of prestressed concrete bridge can also be carried out by CFRP.

5 Conclusion

Confidence of construction industry for long term use of FRP composites has not developed as yet. It is very important that a pre-design study for the long time durability of FRP structures, should be carried out. In order to assess, the durability of FRP structures, it is necessary to have a fair idea of the various chemical constituents present in the environment, their pH, etc. Considering the importance of the structure and the likelihood of salinity, alkalinity, and acidity in the air, some precautionary measures need to be well studied and implemented.

From available literature, general ideas about these materials, their behaviour over a long period of time, it is felt that acceptance of FRP material in primary load bearing structure may not be immediately accepted by the construction engineers.

6 Acknowledgements

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7 References

ASM International (1989), Engineered Materials Handbook – Composite, Vol. I.

Hancox, N.L. and R.M. Mayer (1994), Design Data for Reinforced Plastics, Champman and Hall.

Iyer, S.L. and V. Kalyanram, Eds. (1995), Fibre Reinforced Structural Plastics in Civil Engineering.

Kaushik, S.K., G.L. Asawa and A.K. Ahuja, Eds. (1996), Civil Engineering Practices, Vol. I and II, New Age International Publishers.

Mayer, R.M. (1985), Material Selection and quality Control of Glass Reinforced Plastics, Proc. 7th British Wind Energy Workshop, Oxford, MEP, London.

Pritchard, G. and S.D. Speake, (1987), The use of water absorption kinetic data to predict laminate property changes, Composites 18: 227-232.

Richardson, T. (1987), Composites: A Design Guide, Industrial Press.

Shenoi, R-A. and J.F. Wellicome (1993), Composite Materials in Maritime Structures, Vol. I, Cambridge University Press.

130 FIELD STUDIES CONCERNING THE SERVICE LIFE PREDICTION

I. Gosav

Concrete, Materials and Technology Department, Construction and Architecture Faculty, Technical University "Gh. Asachi", Iasi, Romania

Abstract

The service life prediction for the reinforced and prestressed concrete cross sections subjected to carbonation, chloride, sulfate and nitrate attack, of the reinforced and prestressed concrete building member and structure is presented on the ground of field data collected from the assessment and surveying of more than one hundred constructions. The parameters, which have to be taken into account, are discussed and some examples are given.

Keywords: Aggressive attack, member, RC/PC cross section, service life, structure

1 Introduction

The service life of the existing structures represents an essential parameter, which must be considered when a decision has to be done regarding the future use of these structures.

If the service life of a structure is defined as the period of time between the designing and putting into operation and the moment when an accepted safety level is exceeded, few concepts have to be accurately specified. These concepts could be discussed on a well-known (CEB 1983) schematic representation of the service life, shown in Fig. 1.

1. Firstly, it has to be accepted that a structure *is born* with some ***initial damages,*** which depends on the knowledge of the designer and the performing practice of the period.
2. Then, the building *age* and the ***rate of degradation*** could describe the process. This parameter depends on the characteristics of the building

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materials, the aggressiveness of environment and the protection level of building members.

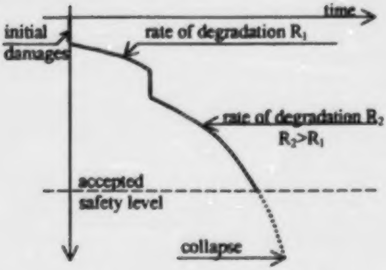

Fig. 1: Schematic representation of the service live of a member / structure

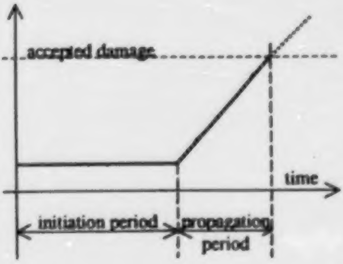

Fig. 2: The service life for a RC/PC cross section

3. During the lifetime of the building, some *accidents* like an ***oversized load*** (an earthquake, for example) could appear and damage it suddenly. Usually, after this, the rate of degradation increases.
4. Finally, the ***accepted safety level*** has to be defined for each member of the structure and for the entire structure, too.

Mainly, the evaluation of the service life consists of the evaluation of the four-presented parameters for:

- The **materials** which make up the cross section of the building members,
- The **cross sections** of the members resulting from the association of these materials,
- The **members** with different composition and loaded cross sections and
- The **structure**, where each member and its connections participate differently to the safety level.

Actually, the research is focused on the durability of the building **materials** for which, knowing the ***rate of degradation*** means knowing the service life, and models of the degradation processes have been found. Several methods to predict the service life of concrete and reinforcement (as separate materials) and reinforced concrete (as an associate material into an unloaded building member cross section) were grounded for different aggressive environments (Clifton and Knab 1989; Clifton 1993): (a) assessment grounded on the expert experience; (b) comparison with similar situations; (c) using accelerated tests; (d) mathematical and empirical models for degradation processes; (e) using reliability and stochastic analyses.

An important finding resulting from the field studies is that, in practice, the application of these methods could often give inconclusive results, because of the multitude of parameters, which have to be considered. Regarding the material, the main parameters are the chemical components and the porosity, which can be determined from samples extracted from building members. Even so, it is quite difficult to establish the initial structure of the concrete (type of cement, water/cement ratio and the grading of aggregate) and its long-term changes. Many times, the ***initial damages*** were decisive for the in service behavior of the materials. Regarding the environment, the most important parameters are the type of attack, the concentration of the aggressive agent, the exposure time and the moisture variation. The problem is that their variation during the service life of the building is very difficult to establish and so is the rate of degradation.

For composite materials such as the reinforced and prestressed concrete, few models concerning the service life prediction are conceived. Tuutti's model (Tuutti 1982) show in Fig. 2 is the classic example. In this model, the initiation period, the first part of the service life, can be defined as the time period up to the complete damage and/or penetration of the concrete cover by the aggressive agent. The remaining time period up to the damage of the reinforcement could be considered the propagation period if the RC **cross section** is unloaded. For a loaded RC cross section, the time up to reaching a certain accepted bearing capacity resulting from the decreasing concrete and reinforcement areas has to be considered as the propagation period.

For a building **member**, with different composition and loaded cross sections, the assessment of the service life becomes more difficult because, in addition to the previous parameters, some other parameters have to be considered. The extension of the damaged area of reinforcement and/or concrete, the anchorage of the undamaged steel bars in the sound concrete and the balance between the stress and the bearing capacity of the damaged cross sections are the main parameters.

The service life of a **structure** depends on the service life of each member, but its participation in the strength and stability of the structure has to be considered. In a simple way, the service life of the most important/loaded/damaged member (foundation, column and wall) could represent the service life of the structure, but a probabilistic or stochastic approach could be done too.

Finally, the service life prediction remains an assumption because it is based on field and experimental studies, almost never confirmed by practice because nobody surveys and waits for the failure of a building member or structure.

Taking into account that the chemical processes within the steel and concrete (CEB 1989) are well known, some useful principles and rules to assess the service life of reinforced and prestressed concrete cross sections, members and structures under different chemical attacks were shown further as a result of the field studies.

2 The service life prediction of the reinforced and prestressed concrete

2.1 To carbonation

The time up to the complete carbonation of the concrete cover could be considered the initiation period for the reinforced concrete cross sections subjected to carbonation. The phenomena has been studied and many models were elaborated, but the practical assessment of the initiation period has to be made for each real case because the results are very scattered (see Fig. 3). Knowing the exposure time and the corresponding carbonation depth, enables determination of the depth of cover carbonation. The test for determining the carbonation depth is one simple test.

The propagation period starts after cover carbonation, but its end has to be defined. The appearance of cracks due to reinforcement corrosion is the first sign that the bearing capacity of the cross section begins to decrease, however time failure is a long way off. Experimental studies (Gosav 1998) show that, for a reinforced concrete cross section, the real breaking load is 2...3 times larger than those computed using the design values of the material strengths.

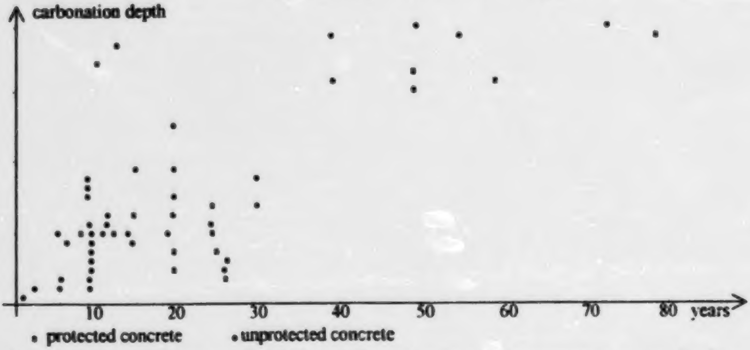

Fig. 3: The carbonation depth for different ages of the concrete as results from field tests

A certain damage percentage of the reinforcement area could be accepted and the time period necessary for this degradation to occur represents the propagation period.

The remark is not valid for the prestressed cross sections, where the smallest decrease in the tensioned reinforcement area could produce failure. Consequently, only the initiation period has to be considered as the service life of a prestressed concrete cross section.

2.2 To chloride attack

The diffusion time of the chloride ions through the concrete cover could be considered the initiation period. The diffusion coefficient is the parameter that describes the phenomenon and it could be measured on sample extract from the building members (Clifton et al. 1991). Taking into account that the concrete components bind a certain percentage of the chloride ions, for reinforced concrete cross sections, the initiation period could be extended with the necessary time that this process requires. Therefore, the free chloride contents in concrete, near reinforcement, could show the border between the initiation and propagation period. The free chloride test could be made in situ using adequate equipment.

Similar to carbonation, the end of the propagation period for chloride attack could be the moment when a certain percentage of the reinforcement cross section is damaged.

For prestressed concrete cross sections subjected to chloride attack, because pitting corrosion can take place before the binding chloride ions by the concrete components, to consider the initiation period as the service life is rational.

The degradation of reinforced concrete due to chloride action depends on environmental parameters and concrete characteristics. The field observations demonstrate that an accurate correlation between these two is very difficult to establish in practice, so to improve the service life assessment, a long term survey is necessary. A relevant example is illustrated in Table 1, where the exposure conditions and the chemical characteristics of concrete exposed for ten years in a saline environment (in a salt exploitation site) are shown (Gosav 1988).

As shown in figure 4, in a dry environment, salt dust covers the element but does not damage it (the first element from Table 1). The tests demonstrate that the moisture variations affect the damage amplitude much more than the chloride contents from the atmosphere and from the concrete. This fact clearly results if the exposure conditions and the chloride contents for the elements 4 and 2 from Table 1 are compared. The damages of these elements are shown in Fig. 5 and Fig. 6.

The cement proportion and the compactness of the concrete (actually, the concrete class) are important parameters too as it results comparing of the chloride contents in elements 2 (usual concrete class) and 3 (high concrete class) from Table 1.

Table 1: Exposure conditions and concrete components in a saline environment

Tested element	Exposure conditions					Concrete components				
	Moisture Conditions	Fix dust residue (%) at105°C	Cl^- (mg/l)	Na^+ (mg/l)	pH	Cl^- (%)	Na^+ (%)	Ca^{++} (%)	CaO (%)	pH
1 Inside column (fig. 4)	dry, without moisture variation	60	40.8	19.3	5.8	1.77	42.18	124.5	*	9.2
2 Outside column (fig. 6)	climatically annual and daily moisture variation	760	459.7	267.1	5.8	3.63	11.91	146.3	*	7.5
3 Outside prestressed beam with bonded tendons (fig. 7)						1.24	32.20	152.7	*	7.6
4 Inside column (fig. 5)	Daily large moisture variation. Vapor with salt (not dust)	93	55.0	35.65	7.3					
0…1cm width						1.24	*	*	13.3	7.7
0…2cm width						1.24	*	*	15.22	7.8
0…3cm width						1.45			11.90	7.9
Rust of reinforcement						0.88	*	*	-	7.9

* Not performed analyzes

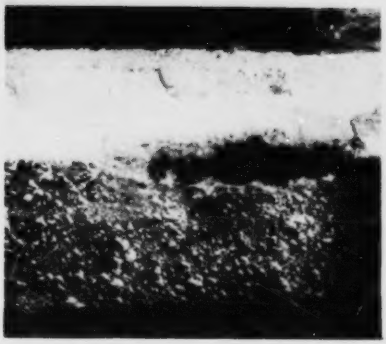

Fig. 4: Salt dust does not damage a reinforced concrete member, in a dry environment

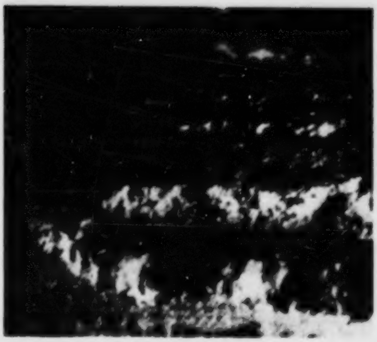

Fig. 5: Deep damage of an element in a large moisture variation saline environment

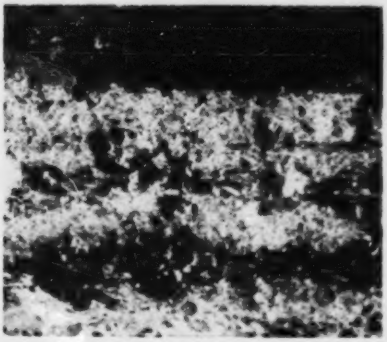

Fig. 6: Damages to an element exposed to the outside saline environment

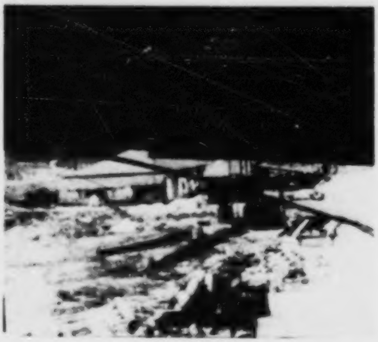

Fig. 7: Damaged tendons to a pestressed concrete element

2.3 To sulfate attack

Because sulfate gradually attacks the concrete through thin successive layers spalling (Fig. 8), the initiation period for reinforced concrete cross sections could be considered the time to complete damage of the concrete cover. It has to be taken into account that the sulfate affected concrete depth is not alkaline as the in field tests demonstrate (Fig. 9). Though this layer is more compact due to the gypsum and ettringite formed in the pores, the reinforcement is still exposed to corrosion because oxygen can diffuse due to the moisture variation. The process is not so dangerous for reinforced concrete cross sections that the previous assumption, that the initiation period finished after the completely damage of the concrete cover, is realistic (Gosav 1990a), unless alkalinity is lost.

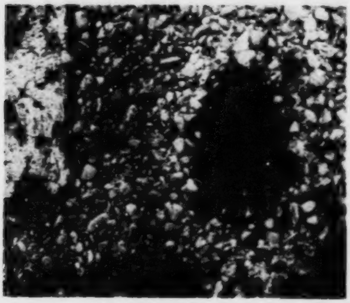

Fig. 8: Sulfate attack of the concrete

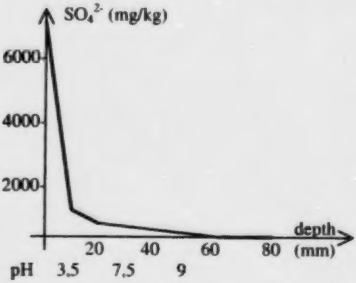

Fig. 9: Sulfate ions penetration profile in concrete

If a certain reducing of the reinforcement area due to corrosion is accepted, as previously, for reinforced concrete cross sections, the end of the propagation period is that necessary to reach this accepted damaged reinforcement area.

2.4 To nitrate attack

Nitrate attack is a combination of the previous two phenomena (chloride and sulfate). The nitrates penetrate, by diffusion, into concrete and disintegrate it on a certain depth (Fig. 10). The reinforcement is affected not only by corrosion, but by an embrittlement phenomenon that takes place (Fig 11), and the reinforcement cross sections are reduced through cracking (Gosav 1990b). An important finding is that the corrosion rate of reinforcement due to nitrate attack is reduced after repassivation of the reinforcement, but further, reduction of the reinforcement cross section area is due to embrittlement. The field and laboratory studies developed until now do not point out the relationship between exposure time and the appearance of the first crack, nor if the cracking takes place gradually or suddenly in the steel bar. However, on the basis of the known field data, it is realistic to accept that the embrittlement is a later phenomenon than the corrosion.

Knowing this, the initiation period for the reinforced concrete cross sections is that necessary for a part of the concrete cover to be damaged, and the rest be penetrated by nitrates. Because of the mechanism of the reinforcement deterioration, both through corrosion and embrittlement, the propagation period has to be more carefully assessed. Assuming that the time up to the cracking of reinforcement due to embrittlement is larger than the period up to the first fine crack appearance in the concrete cover, the second could be considered like the end of the propagation period for reinforced concrete cross section.

For prestressed concrete cross sections, the initiation period has to be considered the time up to nitrate reach the reinforcement through diffusion, so a diffusion coefficient or the concentration of the nitrates at the depth of the reinforcement have to be experimentally found.

Because the embrittlement is clearly more dangerous to the prestressing reinforcement, this initiation period, defined previously, has to be considered as the service life.

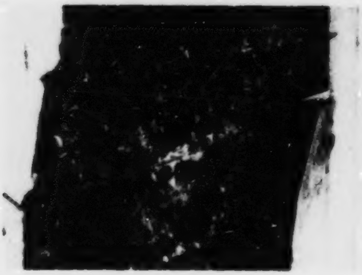

Fig. 10: Damage of concrete and reinforcement due to the nitrate attack

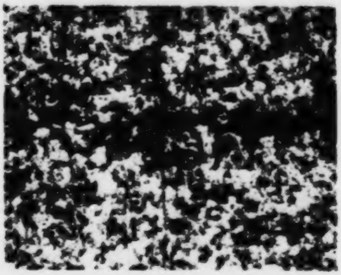

Fig. 11: Intercrystalline crack into the steel reinforcing bar (500:1)

3 Service life prediction for reinforced and prestressed concrete members

3.1 Reinforced concrete members

At first view, the service life of a building member is the service life of that damaged cross section for which the bearing capacity related to the corresponding stress generated by the exterior load exceeds an acceptable value. By knowing the service life of the damaged cross sections predicted as previously shown for different type of attack, the service life of the building reinforced concrete member results. Experimental studies (Gosav 1998) show that this rule could be applied only for locally damaged reinforced concrete beams and columns.

Unfortunately, the problem is not as simple for beams with large exposed reinforcement because some other parameters could become decisive. Experimental studies performed on reinforced concrete beams with exposed reinforcement (Eyre and Nokhasteh 1992; Cairns and Zhao 1993; Raoof and Lin 1993) demonstrate that the following parameter influence their bearing capacity: (a) length of the exposed reinforcement; (b) position of the exposed reinforcement on the member length; (c) depth of the removed concrete over the damaged reinforcement; (d) undamaged anchorage length of the steel bars; (e) bond between the compressed concrete area and the tensioned reinforcement area; (f) percentage of the exposed reinforcement.

A solution consists in imposing some limits to the damage extent so that reaching the limit means the end of the member service life. For example, these limits could be: (a) reinforcement exposure length less than 1/3 from the beam clear span; (b) anchorage length into the sound concrete minimum 50 diameters; (c) debonded bars only on the cover side, the other side being in contact with concrete on minimum half of perimeter. Assuming these conditions, the assessment of the service life of the reinforced concrete members could be made following the rule presented at the beginning.

A question resulting from the field studies is why reinforced concrete members (as in Fig. 5 and Fig. 6) are more damaged than it was assumed previously for the end of their service life, yet do not fail. The answer is that bearing capacity reserves result from the difference between the design values of the materials strengths and the real ones, the plastic adaptability of the reinforced concrete cross sections and members and connections to the bearings and redistribution of the stresses (Gosav, 1998).

A good knowledge and assessment of the damaged elements could lead to a more accurate prediction of the service life. In this stage of the research development, some absolute rules to take into account the damages of these elements can not be given. The experience and the good practice of the expert are integrals.

3.2 Prestressed concrete members

As previously shown, for prestressed concrete cross sections damage of the reinforcement is not accepted and the end of the service life is considered the moment when the entire concrete cover is affected by the aggressive agent, which is dangerous for the tensioned reinforcement. Taking into account that a prestressed concrete member could suddenly fail when a part of reinforcement breaks, its service life has to be more carefully predicted. A maximum safety prediction is to assume that the service life is that of the most affected cross section. But, on a prestressed concrete member, there are quite large areas where the cross sections are over-reinforced (for example, because the same straight tendons reinforced the pin ended girder over their entire length even if the bending moment decreases towards ends). This could be a reason to accept a certain amount of damage of the reinforcement in the over-reinforced cross sections.

It is important to observe that damage of the prestressing reinforcement means the end of service of a tensioned wire and consequently, of a tendon or a cable. It results that the service life of the members with post tensioned reinforcement is that of the most affected cross section, because the breaking of an unbonded reinforcement leads to the decreasing of the bearing capacity for all cross sections. The breaking of a bond tendon means the loss of adherence on certain lengths for both sides of the damaged area and transfer lengths of the stress between the de-tensioned reinforcement and concrete has to be considered (Gosav 1998). All cross-sections along these transfer lengths, both sides of the damaged area, have to be considered as damaged. If a reinforcement damage is accepted, the service life of the remaining cross section is that of a new one but with less reinforcement with the respective to concrete cover.

Taking into account these remarks, the service life of a member with pre-tensioned reinforcement could be considered as the service life of the most affected cross sections from the more loaded areas and the most damaged ones from the less loaded areas. For the cracked prestressed members, the previously shown predictions are not available because the tensioned reinforcement is directly exposed.

Beyond these rational remarks about the service life prediction of the prestressed concrete members, the field studies point to some very interest facts.

For example, a 15m span and 8 tendons reinforced member does not collapse even if one tendon looks like in Fig. 7 and another is corroded after 10 years exposure in a saline environment. Another prestressed concrete member subjected to the same environment, but with initial fine cracks due to under-prestressing, collapses after the same period of time.

4 Service life prediction for reinforced concrete structure

To predict the service life of a structure is a very complicated problem but, simplifying, it could be defined in two ways: (a) as a minimum of the service life of the component members; (b) as a service life for which a characteristic of the structure exceeds an acceptable value.

Concerning the first assumption it has to be taken into account that the members have different contribution in the assurance of the safety level of the structure. A hierarchy, starting with the most important and ending with the least one, is the following: foundation → column and/or wall → girder → beam → slab. Moreover, for each member type, a new hierarchy could be adopted according to the loading level (for example, a central column is less loaded in flexure than a corner one). It results that it is rational to consider the service life of the structure as the service life of the member with the minimum bearing capacity/stress ratio. A more accurate prediction has to take into account the safety reserves resulting from a redistribution of the stresses due to the spatial working manner of a structure and the plastic behavior.

The second manner consists in predicting the time up to that a characteristic of the structure exceeds a certain accepted value. These characteristics could be: (a) stress (e.g. the compression into the column); (b) bearing capacity (e.g. the shear force at the ground level); (c) displacement (the horizontal at the top level); (d) the ductility.

5 Conclusion

Based on field studies and some tests performed on damaged members, some practical rules regarding the assessment of the service life of a cross section, member and structure from reinforced and prestressed concrete, subjected to different chemical attack, are suggested in this paper. The present data are not sufficient for an accurate service life prediction and the experience of the expert is still the most important factor. Only future field data systematically collected and analyzed, correlated to accelerated tests and an extensive experimental program regarding the behavior of the damaged members and structures can improve the service life prediction.

6 References

Cairns J., Zhao Z. (1993) Behaviour of Concrete Beams with Exposed Reinforcement. *Proc. of the Inst. of Civil Eng. Structures & Buildings*, Vol. 99, pp. 141-154

CEB BULLETIN D'INFORMATION No. 162 (1983) Assessment of Concrete Structures and Design Procedures for Upgrading (Redesign)

CEB BULLETIN D'INFORMATION No. 182 (1989) Durable Concrete Structures. CEB Design Guide.

Clifton J. R., Knab L. I. (1989) Service Life of Concrete. NISTR 89-4086, National Institute Of Standards and Technology

Clifton J. R., Knab L. I., Garboczi E. J., L.Xian Xiong (1991) Chloride Ion Diffusion in Low Water-To-Solid Cement Pastes. NISTR 4549

Clifton J. R. (1993) Predicting the Service life of Concrete. *ACI Material Journal*, Vol. 90, No. 6, pp. 511- 617

Eyre J. R., Nokhasteh M. A. (1992) Strength Assessment of Corrosion Damaged Reinforced Slabs and Beams. *Proceedings of the Institution of Civil Engineers. Structures & Buildings*, Vol. 94, May, pp. 197-203

Gosav I. (1988) The Effects of the Chloride Action Against the Constructions Exploited in the Saline Environment. Proceedings of *The in Situ Behavior of the Construction* Symposium, Vol. I, pp. 159-168

Gosav I. (1990) The in Time Behavior of the Industrial Buildings Subjected to the Sulfate Action. Proceedings of *The in Situ Behavior of the Construction* Symposium, Romania, Vol. III, pp. 365-372

Gosav I. (1990) The in Time Behavior of the Industrial Buildings Subjected to the Nitrate Action. Proceedings of *The in Situ Behavior of the Construction* Symposium, Romania, Vol. III, pp. 357-364

Gosav I. (1998) The Residual Bearing Capacity of the Reinforced and Prestressed Concrete Damaged Members. Bulletin of Polytechnic Institute of Ia• i, Vol. XLIV(XLVIII), 1-2, Section VI, pp. 107- 121

Raoof M., Lin Z. (1993) Implication of Structural Damage to Concrete Elements. $2^{\text{-end}}$ International Conference of *Bridge Management Inspection, Maintenance Assessment and Repair*, Thomas Telford, London, pp. 66-74

Tuutti K. (1982) Corrosion of Steel in Concrete. Swedish Cement and Concrete Research Institute, Stockholm

131 EMPIRICAL VS THEORETICAL LIFE PREDICTION FOR SUBFLOOR STRUCTURAL CONNECTORS

P.W. Haberecht and C.D. Kane

Building Research Association of New Zealand, Judgeford, Wellington, New Zealand

Abstract

Using model subfloor enclosures built in a very severe marine environment, corrosivity was measured as a function of four different ventilation regimes. Corrosion-related parameters were measured simultaneously in the same enclosures, and used to determine corrosion rates based on three predictive tools; the ISO 9223, ISO-CORRAG, and MICAT corrosion prediction algorithms. The predicted corrosion rates did not match the observed rates in all cases, with the ISO-CORRAG flat zinc corrosion rate prediction equation providing the closest match within the experimental limits. It is believed that the Time of Wetness (ToW) is a significant factor in the corrosion rates observed, with the actual times of wetness on the specimens being significantly higher than those predicted from humidity measurements. This is due to the presence of chlorides (sea salt) on the panel surfaces. A very good correlation was found between the surface area of the vent and the corrosion rates of zinc and steel, and chloride deposition rates. In New Zealand the severity of the corrosivity in subfloor conditions with restricted ventilation indicates that steel or galvanized subfloor components would not survive the 50 years normally required by the New Zealand Building Code (NZBC).

Keywords: Corrosion rate, Chloride deposition, Marine, Steel, Subfloor, Vent, Zinc

1 Introduction

Galvanised subfloor connectors are normally required by the New Zealand Building Code (NZBC) to be durable for 50 years, but previous studies by Holcroft (1996) have shown that they can be rendered unserviceable by corrosion after a relatively short exposure. Concerns have been raised over the ability of the corroded galvanised components used in the domestic building industry to continue to function

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in severe marine/coastal environments (Cole *et al*, 1996). Subfloor connectors (Figure 1) used in New Zealand are usually made of hot-dip galvanised steel and come in a variety of forms. Traditionally they were galvanised

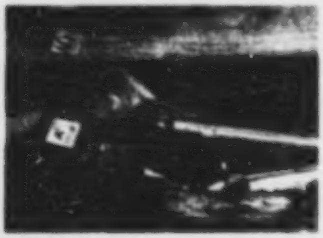

Fig. 1: Common subfloorfittings

wire which was stapled to timber piles and bearers (or looped through, or embedded in concrete piles). More recently, because of increased structural requirements in modern building codes, they have tended to be proprietary nail plate systems, including nail plates, cleats and 'wire dogs'. The most significant factor that causes the corrosion is the prevailing moist salt air from the coastline, and most New Zealand cities are coastal. In New Zealand, a marine zone surrounds both main islands, with a severe marine zone on much of the west coast of both islands (Duncan and Whitney 1982; Kane 1995). Previous studies show that high chloride deposition rates were measured up to 30 kms inland (Ballance and Duncan, 1985), which is substantially further inland than normally expected.

This work compares three predictive corrosion rate models, developed for outdoor environmental corrosivity, against the actual subfloor corrosion rate results, and also evaluates the effect of vent size on subfloor corrosion rates. This provides a greater understanding of the variations in microclimates and corrosion rates that exist in subfloor conditions near coastal environments.

1.1 Previous studies

Many empirical studies have measured environmental corrosivity and developed a predictive equation (Morcillo *et al* 1995; King 1988; Duncan and Ballance 1988). Other studies have been based on measurements of the prevailing environmental conditions using data such as rainfall, temperature, relative humidity, cloud cover and aerosol contaminants (Hyland and Enzensberger 1998; Cramer *et al* 1996; Spence *et al* 1992; Pourbaix 1982; Summitt *et al* 1982).

Subfloor environmental conditions are different from the outside environment, simply because the sub-floor is shielded from rain, and there is some thermal insulation or heating provided by the building above. Depending upon the type of ventilation, there may be limited air movement. Studies by King *et al* 1995, found that the microclimate under sheltered conditions can result in a significant increase in corrosion rates for galvanised steel. They found corrosion rates were 2.4 times greater in severe marine and 2.1 times in moderate marine environments. In contrast, Holcroft (1996) found that components shielded from the prevailing moist salt laden wind were significantly less corroded.

The corrosion process for the exposed zinc surface of galvanised components, on contact with dew, fog, mist or rain, is as follows. Zinc will form zinc hydroxide, which reacts with carbon dioxide from the atmosphere to form an insoluble basic zinc carbonate (Legault *et al* 1978; Cole and Bradbury *et al* 1996). This corrosion product film tends to inhibit any continuation of the corrosion process. The development of the film is a continuous process and it has been found that rain can wash away a substantial amount of the non-adherent zinc hydroxide and expose fresh zinc. This results in further corrosion of the zinc and over a year, the corrosion rate for outside coupons was found to be greater for surfaces exposed to the rain than for surfaces facing the ground (Legault *et al* 1978). However, in environments strongly influenced by the deposition of sea salt, the groundward surfaces of coupons exposed for ten years in New Zealand corroded faster than the skyward surfaces. This was basically because of the influence of accumulated sea salt on the groundward surface, plus a correspondingly increased period of surface wetness or ToW (Haberecht, *et al* 1998), and the corrosion product could be zinc hydroxychlorides instead of the zinc hydroxycarbonates (Cole and Bradbury *et al* 1996).

Sheltered corrosion studies by Norberg *et al* (1996) in severe marine conditions in Australia found that the ToW period was between 5% and 35% greater than fully exposed surfaces and a 10 to 25% increase in zinc corrosion rates had occurred. As part of their experiment, identical zinc specimens were regularly doped with salt solution for a year, at the rate of 50 mg/m^2/day and 500 mg/m^2/day while under the shelter. On average, the corrosion rate was 17% greater for the high salt concentration. Considering the salt concentration was 10 times greater, the corrosion rate increased by a relatively small amount. These results indicate that there is a threshold of surface chloride deposition that has a significant effect on the corrosion rate.

Guttman and Sereda (1968) found that increasing the chloride deposition rate from 10 to 40 mg/m^2/day, and maintaining the ToW at 0.40 (fraction of the year above 80% RH), increased the corrosion rate of zinc by 56%. Their study in Panama also found that increasing the ToW fraction from 0.3 to 0.7, increased the corrosion rate by 79%, regardless of the chloride deposition rates previously mentioned. This indicates that the ToW appears to be more significant once chloride is on the surface.

The ToW period is influenced by the deposition of sea salt on to the surface, which results in a layer consisting of NaCl and $MgCl_2.6H_2O$, where NaCl has an equilibrium relative humidity of 77%, and the $MgCl_2.6H_2O$ is at an equilibrium of 33% (Cole *et al* 1996). This means that if enough sea salt is deposited onto a surface and the ambient conditions are above 33% relative humidity, then the surface will have some condensation present. Evans (1976) noted that the corrosion rate on surfaces contaminated with sea salt increased at 50% RH, and not 33%. The empirical value of 50% RH may take into account the accumulation of enough condensed moisture to actually cause a measurable acceleration of the corrosion process.

Many European corrosion rate models place a heavy emphasis on sulphur dioxide (Haynie 1980; Cramer *et al* 1988). New Zealand has a very low sulphur dioxide concentration and in fact has one of the least industrially polluted atmospheres in the world (Walker *et al* 1996). On this basis the most significant factors that influence the corrosion rate in New Zealand are most likely to be the ToW and chloride deposition rates.

2 Experimental

This study investigated 12 months of salt deposition in subfloor conditions in a specially constructed facility on the south coast of Wellington, New Zealand, at Oteranga Bay as shown in Figure 2. The facility comprised eight adjacent enclosures, each with identical dimensions, replicated subfloor conditions open to onshore winds, as shown in Figure 3. Each enclosure measures 1.2m (W) x 1.2m (H) x 3.6m (L) with the south end facing the sea, and is fully fibre-cement sheet lined, including the roof. The roof of the structure has a minimum air gap, open to the environment, of 100mm between the galvanised steel roof and the fibre-cement sheet lining.

The surface area of the ventilation was based upon NZS 3604 (Standards New Zealand 1990). This requires a minimum subfloor ventilation area of 0.0035m^2 per m^2 of floor area. The floor area of each enclosure was 4.32 m^2, requiring a minimum ventilation surface area of 0.00756 m^2 for each end of the enclosure, and this was exceeded as noted in Table 1.

Table 1: Types of ventilation at each enclosure and total surface area at each end

Enclosure	Vent surface area (m^2)	Description of ventilation (enclosures same at both ends)
O	1.4400	Open
4S	0.1320	4 slotted gaps each 1100 x 30 mm
PB	0.0078	'Post box' slot 30 x 260 mm
C	nil	Closed

The corrosion rate studies (weight loss) used two 100mm x 150mm Z275 coil galvanised steel coupons and two mild steel SAE1010 coupons in a vertical rack at the middle of each enclosure. The backs of the steel and galvanised steel coupons were sealed with wax so that one exposed surface faced either the seaward or the landward ends of the enclosure. Similar mild steel coupons were also used to measure the corrosion rate as a function of distance from the vents and were attached horizontally to stakes 100mm above the ground, spaced at 600mm intervals. The stakes were numbered from the seaward end of the enclosure, with coupon number #1 being closest to the sea.

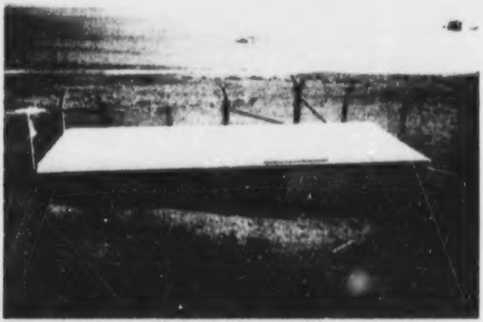

Fig. 2: The eight enclosures with interchangeable vent panels

The environmental conditions were also measured inside each enclosure, 300mm from the internal roof using a calibrated Phillips humidity probe and thermocouple with data logging every 15 minutes. In addition, four salt candles, as described in International Organisation for Standardisation ISO 9225 (1992), were used to determine the chloride deposition rate every 28 days for a year at four positions, as noted Table A2. The salt candles placed beside each other near the seaward vent should generally record the same result, as they were positioned the same distance from the vent or opening, to give a measure of within-site variance. In the middle of each enclosure was a shelf spanning the width of the enclosure at a height of 800mm, which provided a site for the middle salt candle and a rack for supporting the weight loss panels. After one year's exposure, all weight loss coupons were analysed in the laboratory by removing the corrosion products with chemical stripping, as recommended in American Society for Testing and Materials (ASTM) G1 1981, and reported in $g/m^2/year$ units.

3 Results

The average relative humidity (% RH), temperature, and the fraction of the year where the humidity exceeds 75 or 80% RH are shown in Table A1. It was common to record periods of 1.00 ie. 100% RH in the wet winter months, and it appears to be rare for subfloors with restricted air flow to have a humidity level below 75% RH. The temperature and some humidity data in the open enclosure were not available for two months, due to a fault in the data logger. The salt deposition rates and corrosion rates are found in Appendix 1.

3.1 Corrosion rates and vent surface area

The volume and speed of the wind in building subfloor environments are linked to the surface area of subfloor vents. By reducing the size of the vent, a reduction of volume and wind speed is expected. This reduces the ability of the wind to carry large (>7.2μm) aerosols (Walker 1996). The overall effect is a significant reduction of salt deposition rates and subsequently the corrosion rates, and this can be observed in Figure 3. It is interesting to note that the equation of the plot is a simple power law, and this is the same as the common corrosion rate law ($C=At^n$), where C is weight loss, t is time, a & n are coefficients where n is often in the 0.5 to 1 range (Espada *et al* 1996). The results and the equations derived should be considered as indicative and not conclusive, due to the small range of vent surface areas evaluated.

3.2 Three predictive equations

This work investigates three predictive models, which are based on climatic and environmental (pollution) measurements, and include the ISO 9223 (1992), the ISO CORRAG (Knotkova *et al* 1995), and the MICAT (Morcillo 1995) equations. These three equations were selected because they represented a substantial amount of data from around the world on zinc exposed in the open. Since limited investigations have been carried out on predicting subfloor rain-sheltered corrosion rates, it was of interest to compare the common corrosion predictive (outdoor) equations against subfloor (sheltered) conditions. In the ISO CORRAG and the MICAT equations, the 50% RH value was included because Evans 1976 noted an increase in corrosion rates on surfaces coated with sea salt at 50% RH and the main constituent of sea salt is NaCl. The 75% RH was included because water condenses on NaCl at approximately 75% RH and is considered to be a better estimate (Spence *et al* 1992). This may be in dispute since Cole and Bradbury *et al* (1996) found that reducing the humidity from 90 to 60% RH at the high chloride (sea salt) deposition rates (120 mg/m²/d) did not affect the corrosion rate of the galvanised steel. This indicated that in their study there is probably a greater dependency on salt deposition rates than on ToW.

Table 2: ISO 9223 classifications compared to actual corrosion rates of galvanised zinc panels

Site	Classification				
	ToW (% of year)	P (mg/m²/d)	S (mg/m²/d)	C (predicted) (g/m²/yr)	Actual (g/m²/yr)
75% RH					
Open	(76) *t5*	(0) *P0*	(609) *S3*	C5 (30 to 60)	166 or C5*
Four slots	(95) *t5*	(0) *P0*	(26) *S1*	C3 or C4 (5 to 30)	82 or C5*
Post box vent	(97) *t5*	(0) *P0*	(10) *S1*	**C3 or C4 (5 to 30)**	**18 or C4**
80% RH					
Open	(59) *t4*	(0) *P0*	(609) *S3*	C5 (30 to 60)	166 or C5*
Four slots	(87) *t5*	(0) *P0*	(26) *S1*	C3 or C4 (5 to 30)	82 or C5*
Post box vent	(90) *t5*	(0) *P0*	(10) *S1*	**C3 or C4 (5 to 30)**	**18 or C4**

Note 1: The bold highlights the predicted results that matched the experimental results.

Note 2: The values were obtained from Appendix 1, Tables A1 to A4.

*The predicted corrosion rate is greater than the upper classification limit of 60 g/m²/yr for C5.

3.2.1. Equation 1: ISO 9223 Procedures Reference: ISO 1992

This method integrates the major factors involved in the corrosion process. The factors are ToW (t), pollution (SO_2) deposition (P), chloride (salt) deposition (S), and provides an estimated corrosion rate (C) range. The result in Table 2 is not a quantitative result, but a classification of the severity of the corrosion hazard at the location monitored. The results give an expected range of weight loss for each classification only. The input ranges for the model are found in ISO 9223, with level '5' being the upper limit.

3.2.2 Equation 2: ISO CORRAG (Outdoor) Reference: Knotkova *et al* (1995)

$$\text{CorrLoss}_{\text{[illegible]}} = a_1 + B_1(SO_2) + B_2(ToW) + B_3(Cl) = 0.2098 + 0.15(ToW) + 0.31(Cl)$$

Table 3: Results of ISO CORRAG equation predictions against measured corrosion rates for galvanised steel

Site	ToW (%/year)	SO2 (mg/m²/d)	Cl (mg/m²/d)	Predicted Corrosion rate	Actual Corrosion rate (g/m²/yr)
Open*	76	0	609	**200**	166
Four slots	95	0	26	23	82
Post box vent	97	0	10	**18**	18
Open**	98	0	609	**204**	166

* At 75% RH the surface is wet when NaCl is present (Spence *et al*, 1992).

** At 50% RH the surface is wet when sea salt is present (Evans 1976).

The coefficients (a_1, B_{1-3}), were obtained from Knotkova *et al* (1995). In Table 3, comparing the ToW period for the surface at 75% RH against the 50% RH, resulted in the ToW increasing from 76% to 98%, but had a minimal effect on the predicted corrosion rate.

3.2.3 Equation 3: MICAT (Outdoor) Reference: Morcillo (1995)

$$C_{Zinc} = a_1 + B_1(ToW) + B_2(Cl) = 0.03 + 2.73(ToW) + 0.017(Cl)$$

Table 4: Results of MICAT equation predictions against measured corrosion rates for galvanised steel

Site	ToW (%/year)	SO2 (mg/m^2/d)	Cl (mg/m^2/d)	Predicted Corrosion rate	Actual Corrosion rate (g/m^2/yr)
Open*	76	0	609	218	166
Four slots	95	0	26	260	82
Post box vent	97	0	10	265	18
Open**	98	0	609	278	166

* At 75%RH the surface is wet when NaCl is present (Spence *et al*, 1992).

** At 50%RH the surface is wet when sea salt is present (Evans, 1996).

Note: The coefficients (a_1, B_{1-2}), were obtained from Morcillo 1995.

4 Discussion

The corrosion rate of zinc is predominantly affected by the time of wetness the surface experiences and the environmental pollutants that arrive on the surface. The period of surface wetness is substantially extended by the presence of sea salts on the surface. The continual flow of moist, salt laden air over the zinc provides both water and salt. Increasing the vent size allows a greater flow of air and an increase in the corrosion rate was noted, as shown in Figure 3. The corrosion products that develop in sheltered areas may not form the insoluble barrier film that has been seen to reduce the zinc corrosion rate on unsheltered coupons. In addition, the lack of rain to physically remove the accumulated corrosion products and other chemically aggressive agents, such as salt, is also responsible for increasing the corrosion rate. A predictive relationship was determined for the New Zealand coastal conditions for zinc and steel.

On reviewing the predictive corrosion rate equations in Figure 3, it appears that the corrosion rates for steel and zinc are very similar, with coefficients of 0.500 and 0.357 respectively. On further inspection, the first section (Part 1) of the chloride deposition rate has a coefficient of 0.338, which is very close to that of zinc. This indicates that there is a very good predictive relationship between the chloride deposition rate and the corrosion rate of zinc. This result compares favourably to a previous study by King *et al* (1995), for sheltered specimens in Australia, at nearly the same latitude. The King *et al* (1995) study identified a strong linear relationship between chloride deposition rates and the corrosion rate of zinc. By substituting the largest (average) zinc corrosion rate measured in this study of 134 g/m^2/yr (Table A4), into the derived linear equation $Corr_{Zn} = -6.3616 + 2.7312(\text{Chloride deposition rate})$ from King *et al* (1995), a predicted airborne chloride deposition rate of 51.4 mg/m^2/d was obtained. This is substantially less than the measured airborne chloride deposition rate of 609 mg/m²/d recorded in this study. This indicates that an upper threshold for chloride deposition exists and on exceeding the threshold, the corrosion rate generally does not increase, which is similar to the results of Norberg *et al* (1996).

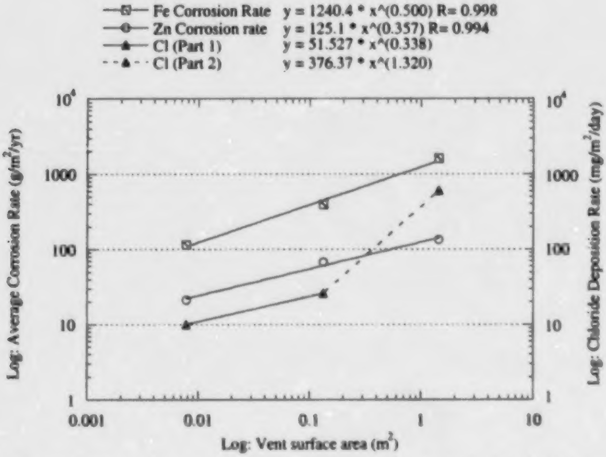

Fig. 3: **Log plot of vent surface area (X) Vs Average corrosion rate (Y_1) and Chloridede position (Y_2), including simple power law equations**

The environmental conditions present in the different subfloor enclosures varied significantly. Comparing the vent surface area (Table 1) to the environmental conditions (Table A1) indicated that a significant increase in the subfloor average humidity occurred as the surface area of the vent decreased. The average temperatures in the enclosures however, were generally less than outside conditions, by up to 2.7°C and surprisingly, the subfloor temperature was not strongly influenced by the type of ventilation. The resulting environments in the four slot and post box slot enclosures were colder and recorded ToW periods exceeding 95% of the year. There were periods in these two enclosures where the surface would have remained continually wet for many days. The open-ended test facilities and the general atmospheric conditions generally recorded similar conditions, which indicates that the high air flow in the open enclosure maintained an equilibrium between that subfloor and the external environment.

The salt candle analysis provides a better understanding of the movement of aerosol salt in the subfloor. Table A2 shows that salt candles placed at the ends of the test facilities, nearest to the sea, provided the highest chloride concentrations, which exceeded 1100 mg/m²/d for the open-end test facility. The significant result was the 49% reduction in the chloride deposition rates half way along the 3.6m facility. This reduction can be accounted for by variations in the aerosol size, and a recent study in New Zealand (Walker *et al*, 1996) found that at the coast a fine aerosol and a coarse aerosol were present. They found that 40% of the aerosol was >7.20μm and 50% of the aerosol was between 0.49μm and 7.20μm. In previous studies, MacDonald (1982) in Walker *et al* (1996), noted that aerosols greater than 20 μm contributed only 13% of the total aerosol mass, but contributed 70% of the deposition. This indicates that large aerosols may constitute the majority of salt available for deposition.

The volume of air and the wind speed in the subfloor space will strongly influence the quantity of aerosol and the size of the aerosol present. If the vent reduces the wind speed or possibly introduces some turbulence, then the large aerosols would deposit in a relatively short distance from the vent. The enclosure and the vent may have an effect since Spence *et al* (1992) noted that the mass transport properties of the air and aerosol could be affected by resistance factors, which create friction and ultimately change a laminar flow into a turbulent flow, which results in significantly higher deposition rates of the salt. The present study found that from the front of the enclosure to the rear, a total of 3.6m, the average chloride deposition rate decreased from 1187 to 338 mg/m^2/d, a reduction of 72% (Table A2). This result accounts for the relatively high corrosion rate and chloride deposition rates found near the vent (Table A3), and the measured effect of varying the vent surface area (Figure 3). This effect was noted in a previous study in New Zealand by Holcroft (1996), where the subfloor space of a beach front building was inspected. A significant reduction in the corrosion present on the 40-year-old hot-dipped steel subfloor components was found, as the distance from the sea increased. It was also noted that the components nearest to the sea had already failed, with substantial flaking red rust present.

The comparison of coupons facing seaward and landward (Table A4) indicated that generally there was a small variation due to the orientation of the coupon, particularly for zinc, which compares favourably to a previous study by Cordner *et al* (1990).

The measured corrosion rates were generally very high, and in the open-ended facility the rate was in the range of 102 to 166 g/m^2/year. This rate would reduce the lifetime of typical hot-dip galvanised components (400 g/m^2 of zinc as per AS1650) to well below five years. In the slotted enclosures and Post Box slot enclosures, the corrosion rates were significantly lower, but would last perhaps 10 to 40 years respectively. This expectant life span is not the minimum 50 years as required by the NZBC. The estimates assume that the usual corrosion rate of zinc is linear from the first year, which is not valid for outdoor exposed zinc (Zoccola *et al* 1978). It may be valid for subfloor conditions considering the inability of the surface to be cleansed of the corrosion product or contaminating salt, which strongly influences corrosion rates.

It would be difficult to guarantee a 50-year life for hot-dip galvanised steel components used in the subfloor in severe marine environments in New Zealand, unless they were additionally protected in some other way. It follows that the use of stainless steel (type 304 or preferably 316) is worth considering as an alternative to galvanised steel for very severe coastal locations. If stainless steel were used, with a suitable maintenance/cleaning programme, the ventilation of the subfloor may be increased without compromising the connectors' durability, and subsequently gaining the benefits of a drier subfloor space. A solution, of course, is to minimize all ventilation but this will cause other problems to the timber foundations, such as increasing timber moisture content, increasing the potential for fungal decay, and rising damp. A method to reduce the ingress of moisture into the subfloor is to apply a damp-proof membrane (DPM), isolating the ground from the subfloor air space and reducing the ToW. This method does not isolate the foundations from the soil, which may cause the foundation uprights to become moisture wicks in the subfloor space. The balance between ventilation and using a DPM needs further investigation.

The three equations used to predict the performance of the galvanised steel components, although not designed for sheltered exposure, included ISO 9223, ISO CORRAG, and MICAT. In Table 2, the ISO 9223 methodology generally underestimated the corrosion rate, simply because of the limitations of the ISO 9223 classifications. The ISO 9223 procedure failed to sufficiently predict the corrosion rate where high humidity and high chloride deposition rates occurred.

The ISO CORRAG equation for flat zinc coupons (Table 3) is based on a substantial number of fully exposed studies and predicted the corrosion in the open-ended and the most shielded post box slot exposure quite well. The equation failed to predict the corrosion rate in the four slots (S4) vent enclosure. On reviewing Equation 2, the bias in the equation is placed on chloride deposition rather than ToW. The results obtained in this work indicate that high ToW levels and low chloride deposition rates, result in a high corrosion rate, which is an effect Guttman and Sereda (1968) noted. This again supports the concept that although corrosion rates can be increased with increasing chloride deposition rates, as noted by King *et al* (1995), there is a chloride deposition upper limit, above which corrosion rates do not increase. Significantly greater corrosion rates occur when ToW levels are increased in the presence of pollutants, such as sea salt.

The last predictive equation was the MICAT, which was based on South American data and failed to predict any of the results obtained empirically, as shown in Table 4. The failure was due to an excessive reliance on ToW measurements and in fact the ToW for the sheltered subfloor spaces was extremely high and invalidated the equation.

5 Conclusions

The ISO 9223 and MICAT predictive corrosion rate equations (outdoor) failed to predict subfloor corrosion rates, but the ISO CORRAG (flat zinc) equation predicted two of the three environments. The results indicated that relatively high ToW levels and an upper limit to the chloride deposition rate may be the predominant factors in the corrosion of subfloor hardware, and controlling one or both will significantly reduce the corrosion rate.

Very good correlations between the corrosion rate of steel and zinc, a limited range of chloride deposition rates, and the size of subfloor vent areas were found for severe marine conditions and predictive equations were developed.

When the surface area of the vent to the subfloor was reduced, a very high ToW and a significant reduction in sea salt (chloride) deposition occurred. Salt deposition rates were reduced by 50 to 95% halfway (1.8m) along the length of enclosures, particularly with restricted ventilation levels. This is probably due to the early fallout of relatively large sea spray aerosols.

High corrosion rates were recorded in the severe marine subfloor conditions, which indicates that the life expectancy of galvanised steel structural components is less than the NZBC standard requirement of 50 years. The suggested remedies for this are: coat the galvanised steel with a barrier; use stainless steel (304 or 316); or possibly introduce a damp-proof membrane over the floor of the subfloor space, with some ventilation.

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7 References

AS1650 Standards Association of Australia, 1981, *Galvanised Coatings*, Sydney.

ASTM G1, 1981, *Recommended practice for preparing, cleaning and evaluating corrosion test specimens*, American Society for Testing and Materials, Philadelphia.

Ballance J.A., Duncan J.R., 1985, *Wind-borne transport of sea salt in New Zealand*, New Zealand Journal of Technology, Vol 1, Wellington, pp239-244.

Cole I.S., Norberg P., Ganther W.D.,1996, *Environmental factors promoting corrosion in building microclimates*, Proc13th International Corrosion Conference (ICC), Melbourne.

Cole I.S., Bradbury A., Neufeld A.K., Sherman N.,1996, *Response of galvanised steel, 55% aluminium zinc-coated steel and copper steel to well defined salt doses under controlled environments.*, Proc13th International Corrosion Conference (ICC), Melbourne.

Cordner R.J., Duncan J.R., Krouse D.P. ,1990, *Assessment of atmospheric corrosivity in Taranaki, New Zealand*, Corrosion Australasia, Vol 15, N01, pp11-13.

Cramer S., Carter J.P., Linstrom P.J., Flinn D.R., 1988. *Environmental effects in the atmospheric corrosion of zinc*, ASTM STP 965 Degradation of metals in the atmosphere, Eds Dean S.W., Lee T.S. American Society of Materials and Testing: Philadelphia, pp229-247.

Cramer S.D., Covino B.S., Holcomb G.R., 1996, *Cubic model describing the atmospheric corrosion of structural metals*, 13th International Corrosion Congress, Melbourne, Nov., pp32/1-8.

Duncan J. and Ballance J., 1988, *Marine Salts Contribution to Atmospheric Corrosion*, ASTM STP 965, Ed Dean S.W , American Society of Materials and Testing: Philadelphia, pp316-326

Duncan, J and Whitney, R, 1982, *Suggested zones of steel corrosion hazard in New Zealand atmospheres*. IPENZ Transactions, Vol 9. No 3, pp65-75.

Espada L., Vazquez M.E., Sanchez A., 1996, *Acid rain: It's influence on corrosion processes*, Proc. 13th International Corrosion Conference (ICC), Melbourne, Nov 1996, pp10/1-9.

Evans U.R., 1976, The corrosion and oxidation of metals, 2nd vol, Pub. Edward Arnold, London, p250.

Guttman H., Sereda P.J., 1968, *Measurement of atmospheric factors affecting the corrosion of metals*, ASTM STP 435 Metal corrosion in the atmosphere, Philadelphia, pp326-359.

Haynie F.H., 1980, *Theoretical air pollution and climate effects on materials confirmed by zinc corrosion data*, ASTM STP 691 Durability of building materials and components, Eds Sereda P.J., Litvan G.G., American Society of Materials and Testing: Philadelphia, pp 157-175.

Holcroft. G 1996 *An investigation of 62 coastal subfloors for fastener corrosion*, BRANZ Report No.72.

Hyland C.W.K., Enzensberger M.,1998, *Prediction of site-specific steel corrosion rates in New Zealand to assist coatings selection*, Proc. Australasian Structural Engineering, Auckland, NZ, pp835-842.

ISO9223 & 9225 International Organisation for Standardisation, 1992, *Corrosion of metals and alloys - corrosivity of atmospheres - classifications & measurement of pollution*, Geneva.

Kane C.D., 1995, *Atmospheric corrosion survey of New Zealand: Six year exposure results*, IPENZ Transactions, Vol 23, No.1, EMCh, Wellington, pp29-39.

King G.A. 1988, *A corrosivity survey on a grid of sites ranging from rural to moderately severe marine, Part 1*, Corrosion Australasia, Australasian Corrosion Association, Feb 1988, pp5-12.

King G.A., O'Brien 1995, *The influence of marine environments on metals and fabricated coated metal products, freely exposed and partially sheltered*, ASTM STP 1239 Atmospheric corrosion, Ed Kirk W.W. and Lawson H., American Society of Materials and Testing: Philadelphia, pp167-192.

Knotkova D., Boschek P., Kreislova K., 1995, *Results of ISO CORRAG program: Processing of one-year data in respect to corrosivity classification*, ASTM STP 1239 Atmospheric corrosion, Ed Kirk W.W. and Lawson H., American Society of Materials and Testing: Philadelphia, pp39-55.

Legault R.A., Pearson V.P., 1978, *Kinetics of the atmospheric corrosion of galvanized steel*, ASTM STP 646 Atmospheric factors affecting the corrosion engineering metals, Ed Coburn S.K. ASTM.

Haberecht P.W., Kane C.D., Meyer S.J., 1998, *Environmental corrosivity in New Zealand: Results after 10 years exposure*, submitted for publication.

Moricillo M., 1995, *Atmospheric corrosion in Ibero-America: The MICAT project*, ASTM STP 1239 Atmospheric corrosion, Ed Kirk W.W. and Lawson H., Philadelphia, pp257.

Morcillo M., Simancas J., Feliu S., 1995, *Long-term atmospheric corrosion in Spain: Results after 13-16 years of exposure and comparison with worldwide data*, ASTM STP 1239 Atmospheric corrosion, Ed Kirk W.W. and Lawson H., Philadelphia, pp316-326

Norberg P., King G.A., O'Brien D.J., 1996, *Corrosivity and microclimate measurements in open and sheltered marine environments*, Proc. 7DBMC, Stockholm, Sweden, Vol 2, pp181-190.

NZBC, 1992, Building Industry Authority, *New Zealand Building Code*, Wellington.

NZS 3604 Standards Association of New Zealand, 1990, *Code of Practice for Light Timber Frame Buildings Not Requiring Specific Design*, Wellington.

Pourbaix M.,1982, *The linear bilogarithmic law for atmospheric corrosion*, Ed. Ailor W.H., Atmospheric corrosion, John Wiley and sons, New York, pp107-121.

Summitt R., Fink F.T., 1982, *The USAF corrosion testing program and a corrosion severity index algorithm*, Ed. Ailor W.H., Atmospheric corrosion, John Wiley and sons, New York, pp245.

Spence J.W., Haynie F.H., Lipfert F.W., Cramer S.D., McDonald L.G., 1992, *Atmospheric corrosion model for galvanized steel structures*, Corrosion , Vol 48, No.12, NACE, Philadelphia, pp1009.

Walker C.F., Harvey M.J., Boyd I.S., 1996, *Background and urban aerosol chemistry in New Zealand*, National Institute for water and atmospheric research publication, Wellington, New Zealand.

Zoccola, J, Townsend, H, Borzillo, A and Horton, J, 1978, *Atmospheric corrosion behaviour of aluminium-zinc alloy-coated steel*, Atmospheric factors, ASTM STP 646, Philadelphia, pp165.

APPENDIX 1

Table A1: Summary of temperature and humidity results from the test site

	OUTSIDE(1)	OPEN ENDS(2)	FOUR SLOTS(1)	POST BOX SLOT(1)
Max Temp	32.1	29.6	27.7	30.9
Min Temp	7.0	5.5	4.2	6.6
Average Temp	17.8	16.4	15.1	17.4
Temp Stdev	2.0	1.9	2.6	2.5
Average RH%	81.7	82.6	87.9	89.3
RH% Stdev	4.2	6.2	3.5	3.3
≥75% RH (Yr)	0.72	0.76	0.95	0.97
≥80% RH (Yr)	0.60	0.59	0.87	0.90

1) Excludes Jan Temp and all Feb results.

2) Excludes Jan Temp and all Feb results, Aug Temp and some % RH, Sept Temp and % RH.

Table A2: Complete results of salt deposition rates measured at Oteranga Bay

Average Salt Deposition Rate mg/m^2/day	HORIZONTAL PLATE	OPEN ENDS	FOUR SLOTS	POST BOX SLOT
Seaward vent	--	1187	355	279
Seaward vent	--	1186	348	139
Middle of enclosure	--	609	26	10
Landward vent	--	338	21	6
Max deposition seaward end	2130 (May)	4187 (Jun)	1076 (Jun)	857 (Jun)
Min deposition seaward end	92 (Oct)	116 (Dec)	30 (Dec)	12 (Nov)
Horizontal plate average	790	--	--	--

Table A3: Corrosion rate of horizontal steel panels at 600mm spacings in various test facilities

ENCLOSURE	CORROSION RATE (g/m^2/yr)							
	#1	#2	#3	#4	#5	#6	#7	*Average*
Open	2665	2081	1688	N/R	1525	1698	1309	1826
4 Slots	696	670	702	910	603	961	718	751
Post Box	670	605	440	400	447	335	409	472
Closed	206	140	125	220	134	243	318	198

N/R Not recorded

Table A4: Corrosion rate of vertical steel panels at the centre in various test facilities

TEST COUPONS	CORROSION RATE (g/m^2/yr)			
	Open	4 Slots	Post Box	Closed
Steel: Vertical Facing seaward	1605	427	107	60
Steel: Vertical Facing landward	1595	330	124	20
Galvanised: Vertical Facing seaward	166	82	18	31
Galvanised: Vertical Facing landward	102	54	24	29

132 OVERVIEW AND NOTIONAL CONCEPTS ON PERFORMANCE AND SERVICE LIFE

P. Jernberg

Royal Institute of Technology, Gävle, Sweden

Abstract

The representation of various quantities when testing materials and components is quite different depending on the nature of the properties tested as well as the nature of the applied phenomena influencing the material properties. Typically this is found in the domains of mechanical testing vs. that of exposure testing. Nonetheless, these testing domains have much in common and in the domain of testing of long-time performance of building and structure components the, purpose is virtually the same. In many cases both types of test have to be made simultaneously because of synergistic or other types of combined effects which are crucial for the outcome. Obviously, there is a need for a common framework and to sort out the interrelationship between the disciplines. In view of standards and regulations recently developed or under development, addressing durability aspects of building products, this need is further emphasised. This paper discusses this issue, making some observations, and presents an attempt to interpret the concepts of mechanical testing in terms of those of exposure testing, as the latter offers a more general terminology. It also introduces a classification of performance requirements and agents, useful in this context. Furthermore, a common principle for the criterion of the performance requirement, based on the instantaneous performance characteristic is put forward.

Keywords: Performance, service life, building products, exposure testing, mechanical testing, performance requirement criterion

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1 Background

The ways to represent and designate various quantities when testing materials and components are quite different depending on the nature of the properties tested as well as the nature of the applied phenomena influencing the material properties. For instance, when testing materials' and components' ability to withstand mechanical effects, usually the basic concepts are resistance, rupture/fatigue/creep and load, while in the case of exposure to agents originating from the atmosphere, the common corresponding concepts are performance, degradation and dose/intensity, respectively. The reason for this situation seems to be that, on one hand, the two disciplines of testing have evolved from different needs and involve different expertise. On the other hand – although the two corresponding sets of concepts denote the same phenomena in a broad sense – they do comprise different specific features.

Mechanical testing and exposure testing still have much in common, for example what concerns mathematical and statistical treatment, and in the domain of testing of long-time performance of building and structure components the purpose is virtually the same. Furthermore, in many cases both types of test have to be made simultaneously because of synergistic or other types of combined effects which are crucial for the outcome. Obviously, there is a need for a common framework and to sort out the interrelationship between the disciplines. In addition, in view of standards and regulations recently developed or under development, addressing durability aspects of building products, for instance the ISO 15686 series "Buildings – Service life planning" (International Organisation for Standardisation 1998, 1999) and the European Union's Construction Products Directive – CPD (The Council of the European Communities 1988), this need is further emphasised.

Below, in sections 2 and 3 the current view and terminology in the two testing domains is described. In section 4 an attempt to merge the terminology of the two domains is described, carried out by interpreting mechanical testing using concepts and terminology of exposure testing, which choice affords a more general base. A classification of performance requirements and agents, useful in this context, is introduced in section 5. Finally, in section 6 service life and the establishment of performance requirements are discussed.

2 Mechanical testing

The influence of mechanical agents on structural materials and components is usually described by means of the load, i.e. the level of mechanical action, and the resistance, i.e. the ability of the component to withstand the load in terms of a prescribed requirement. In the simplest tests, no attention is paid to the effects of incremental damage, and includes just an observation of the maximum load a component can resist until rupture. In more sophisticated tests, the influence of the effects of incremental damage, such as creep at static loads and fatigue at cyclic loads, is taken into account.

3 Exposure testing

When testing a component's ability to carry out its intended functions, i.e. *performance*, under *exposure* to *agents* originating from the atmosphere causing *degradation*, the performance is expressed in a set of *critical properties* that, in turn, are quantified in terms of a set of *performance characteristics*, each of which have to be maintained above a certain level – a *performance requirement*. When this is no longer fulfilled, *failure* occurs.

The level at which an agent is applied during the test is measured by the *agent intensity*, where "intensity" should be read more generally as compared to a strict physical interpretation (e.g., concentrations and rates are conceptually encompassed in this use of this term).

4 Correspondence of terminology

As already indicated, a number of obvious corresponding concepts between the two domains exist and is shown in Table 1. The concepts of mechanical testing may be viewed as a special case of those of exposure testing. Thus, it would be appropriate that the general case refer to the concepts derived from exposure testing.

Table 1: Correspondence between mechanical and exposure testing

mechanical testing	**exposure testing**
mechanical load	exposure
application of load	
mechanical impact/stress	agent
load level	(agent) intensity
intensity	(mean) frequency
resistance	performance
resistance requirement	performance requirement
strength	critical property
strength level retained	performance characteristic
fatigue/creep	degradation
rupture	rupture
failure	failure (i.e. rupture or degradation)

The designation of mechanical impacts and stresses as *agents* does already have support in International Standards in the building area (International Organisation for Standardisation 1984, 1999). Furthermore, when applying mechanical loads, samples are *exposed* to mechanical agents.

5 Classes of performance requirements and agents

A performance requirement may be classified in one of the following two groups:

- *inherent requirement* — constitutes an appropriate inherent level above which the performance characteristic of a critical property should be maintained, and to which property an agent may be detrimental through a degradation mechanism; and
- *resistance requirement* — constitutes a level of agent intensity that the component should be able to withstand with a sufficiently high probability.

Requirements, such as the level of heat resistance of an insulating material or paint gloss are examples of *inherent* requirements, whereas a requirement on strength of a component is a *resistance* requirement.

For agents, two corresponding groups can be distinguished in accordance with the type of detrimental mode of action they cause:

- *degradation agent* causes gradual decrease in one or several performance characteristics through degradation mechanisms; and
- *rupture agent* may cause an instant rupture, i.e. causing the performance characteristic to drop to zero instantaneously.

Typically an agent originating from the atmosphere, such as UV radiation, is a degradation agent, whereas mechanical agents frequently belong to both groups, as they cause both degradation (fatigue/creep) and may cause rupture, the latter being the ultimate condition of the component. However, for brittle materials, such as glass, mechanical agents often are of a more or less pure rupture nature.

6 Service life vs. performance requirements

6.1 General

The aim of a test or a survey is to predict a service life, defined as the period after installation during which all conditions of a component meet or exceed all performance requirements established. Accordingly, for each of the requirements, the corresponding performance characteristics are recorded during a test or survey. The requirement first found to be violated will then become decisive for the termination of the service life. The critical property reflecting this requirement is designated the *terminating critical property.* This is illustrated in Figure 1, where hypothetical *performance-over-time functions*, i.e. performance characteristics as functions of time, for two critical properties are outlined together with the corresponding requirements (shown as a common level since the performance characteristics scale may be arbitrary chosen). However, in the following discussion, for the matter of simplicity, only a single performance characteristic is considered. For more details on the methodology for prediction of service life, refer to (European Organisation for Technical Approvals 1997; International Organisation for Standardisation 1999; Masters and Brandt 1989).

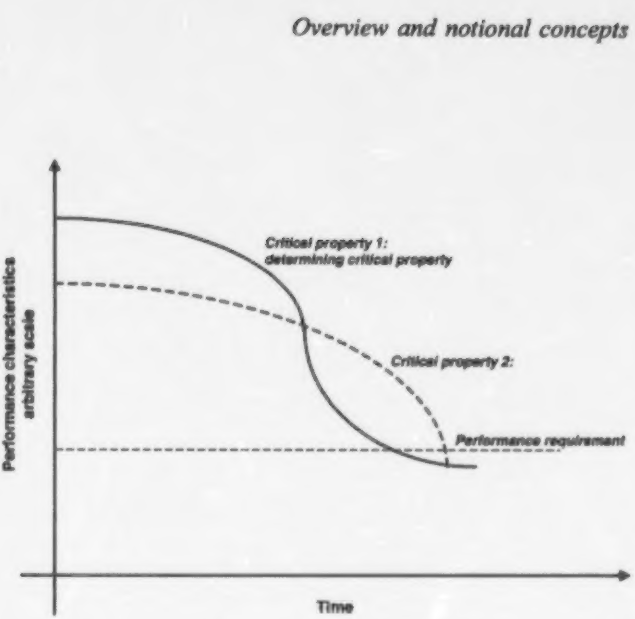

Fig. 1: Performance characteristics as functions of time for two hypothetical critical properties

The performance-over-time function $C(t)$ is assumed to be monotonically decreasing, and the service life t_{SL} is defined by the intercept of the performance-over-time function and the performance requirement level, R:

$$t_{SL} = t \text{ when } C(t) = R \tag{1}$$

That is, to determine the service life, the performance-over-time function has to be known in the exposure situation considered and as well, the performance requirement likewise must be established.

Usually the properties of a set of similar components are stochastic variables. The agent intensities may or may not, depending on the nature of the exposure situation, also be stochastic variables, however, this is the case in regards to ordinary outdoor exposure and of dynamic, stochastic mechanical stress discussed above. Therefore the performance-over-time function $C(t)$ is normally stochastically distributed, as is the time to failure, i.e. the service life t_{SL} .

In the general case, failure, i.e., end of service life, may occur due, either to degradation, in which the value assigned to the performance characteristic is just below the performance requirement, R, or due to rupture, in which case the performance characteristic is zero.

The probability of failure $P_{f_{C<R}}$ between time 0 and time t_{SL} , i.e. the service life distribution, may thus be split into two terms, each of which represent specific modes of failure, i.e. rupture and degradation, respectively:

$$P_{f_{C\leq 0}}[0-t_{SL}] = P_{d_{C\leq 0}}[0-t_{SL}] + P_{r_{C=0}}[0-t_{SL}], \tag{2}$$

where the index, $r_{C=0}$, indicates a rupture and the index, $r_{C\leq 0}$, indicates a degradation failure. The rupture term is explicitly independent of the choice of *R*, whereas it implicitly depends on *R*, since the probability of rupture will increase when leaving low-performing items in service.

As the performance-over-time function is assumed to be monotonically decreasing, the probability of failure $P_{f_{C\leq 0}}$ between time 0 and time t_{SL}, i.e. the service life distribution equals the probability that $C(t_{SL})$ is less or equal to *R*:

$$P_{f_{C\leq 0}}[0-t_{SL}] = P[C(t_{SL}) \leq R]. \tag{3}$$

If the probability distribution of the characteristic *C* at time t_{SL} is denoted $F_C(c;t_{SL})$, then it follows that:

$$P_{f_{C\leq 0}}[0-t_{SL}] = F_C(R;t_{SL}), \tag{4}$$

which expresses the service life distribution as function of the requirement *R* on the instantaneous performance characteristic $C(t_{SL})$.

The distribution function, F_C, may be given in terms of its probability density function $f_C(c;t_{SL})$, yielding:

$$F_C(R;t_{SL}) = \int_0^R f_C(c;t_{SL})dc, \tag{5}$$

Combining eq. 5 with eqs.2 and 4, the following relationships may be identified:

$$P_{r_{C=0}}[0-t_{SL}] = \int_0^{\delta\to 0+} f_C(c;t_{SL})dc \tag{6}$$

and

$$P_{d_{C\leq 0}}[0-t_{SL}] = \int_{\delta\to 0+}^{R} f_C(c;t_{SL})dc, \tag{7}$$

respectively, reflecting the fact that rupture is defined as when the performance characteristic instantaneously goes to zero, mathematically described as a singularity of the probability distribution at $c=0$. This representation provides a formal but compact means to uniquely include both degradation and rupture probabilities in a single distribution.

6.2 Inherent requirements

When rupture does not occur, the second rupture term of eq. 2 becomes zero, and the performance requirements are of the inherent type. These are directly established from needs defined in the design process, in which the subsequent choice of materials and components is made in order to attain an acceptable service life as obtained from eq. 1. When requirements are less rigid in terms of technical performance, requirements and choice of material – and thereby the predicted service life – may be mutually optimised to yield, for instance, minimum life cycle cost or environmental impact. For a description of general design principles in a life time perspective, refer to (International Organisation for Standardisation 1998). An example of inherent requirements is given by Lounis *et al.* (1998a), where a stochastic modelling of the performance distribution of roofing components and systems is proposed for service life prediction, using a discrete Markov chain.

6.3 Resistance requirements

When in a given system rupture may occur in a key component and resistance requirements have to be established, the situation is different. In this case, the actual agent intensity (load level) needs to be considered in order to determine a requirement level such that the material or component will be able to withstand specific intensities over a period of time with a known probability of occurrence. Here it is assumed that the agent intensity is dynamic and stochastically distributed in terms of pulse magnitude, duration and mean frequency, thus defining the probability of occurrence of given intensities. In the field of structural design this situation is well known, where the requirement chosen usually should ensure that the *probability of rupture during the design life* is sufficiently small (Lounis *et al.* 1998b). The more serious the consequences of rupture in terms of life and economical value, the smaller this probability should be, i.e. the safety level should be appropriate for the application. Eq. 2 can express this, but the design life t_{DL} rather than the service life t_{SL} is employed in this instance:

$$P_{f_{C\le R}}\left[0-t_{DL}\right]=P_{d_{C\le R}}\left[0-t_{DL}\right]+P_{r_{C>R}}\left[0-t_{DL}\right], \qquad (2')$$

where the rupture term is subjected to constraint and usually required being very small by virtue of the limitation of probability of rupture during the design life. However, at least for the purpose of service life prediction, this approach is not altogether adequate, as will be evident from the discussion below.

One may observe that – as a component at a given point in time neither is better nor worse than what the performance characteristic at that point of time indicates – the performance requirement should actually be given by the performance level at which the probability of occurrence of a given intensity causes the probability of rupture, that is the *instantaneous probability of rupture*, to become unacceptable (also provided by eq. 1). Accordingly, in this case the *probability of rupture during the service life* is not the formally correct criterion, because the performance history, as for inherent requirements, is virtually irrelevant when determining whether the end of service life has been attained. In fact, the instantaneous probability of rupture would be a more appropriate criterion to employ also in structural design, since maximum probabilities of rupture during the design life as a function of this lifetime is in any case required and prescribed. For instance, two alternative constructions of which one has a design life of ten years and the other of hundred years require quite different probabilities of rupture during the design life. Furthermore, there should be no fundamental reason to use different principles for design life assessment and prediction of service life.

To clarify this standpoint in another more formal way, assume, for simplicity, an ideal construction (e.g. key "component" in a constructed assembly) not subjected to degradation and a load pattern not varying over time. In this case, the degradation term of eq. 2 becomes zero and the failure hazard becomes constant representing an exponential probability distribution. This implies that the service life should only be limited by the occurrence of rupture and that the "component" would exhibit a constant ability to perform. That is, once through the quality assurance approvals, in which the "component" would be deemed to have adequate performance characteristics, this level of performance would be maintained until the "component" eventually ruptured due to a more or less unlikely load event. Obviously, there would be no reason to suggest that the "component" had reached its ultimate service life before it ruptured, irrespective of the total probability of rupture up to the point in time at which rupture occurred, since the "component" is considered, for the sake of this example, "un-deteriorated" up to the time catastrophic failure occurs. So how to determine a performance requirement based on the instantaneous probability of rupture?

At a given point of time, t_1, the instantaneous probability of rupture can be identified as the hazard constant, $h(t_1)$, in a hypothetical process over time where the performance characteristic is constant, $C(t_1)$, and the probability of occurrence of a given intensity is also constant (equal to the probability of occurrence of a given intensity at time t_1). In the particular case where the load intensity pattern follows from a Poisson spike process, characterised by a mean pulse frequency, ν, a random rectangular pulse magnitude, S, with a probability distribution, $F_S[s]$, and a pulse duration approaching zero, this hazard equals the "upcrossing rate" (Lounis et al. 1998b):

$$h(t_1) = F_S[C(t_1)][1 - F_S[C(t_1)]]\nu\,. \tag{8}$$

The performance requirement, R, may now be established by fixing a maximum acceptable hazard, $h_{\max}$, and substituting $C(t_1)$ by R, yielding:

$$h_{\max} = F_S[R][1 - F_S[R]]\nu\,, \tag{9}$$

and then solving for R. Accordingly, if the probability of occurrence of a given intensity varies over time, so would the performance requirement, thus altering eq. 1 to:

$$t_{SL} = t \text{ at first occurrence when } C(t) = R(t). \tag{1'}$$

For simplicity though, it has been assumed that the performance requirement R is constant over time.

7 Conclusions

This paper is an attempt to initiate a discussion that is needed in order to set up a common terminology and methodology in the domains of exposure and mechanical testing, from a viewpoint of exposure testing. In addition to the positive effects from the inevitable unification of the domains, such a discussion will certainly lead to fruitful development in any of them.

Specifically, a correspondence between terms has been identified (table 1), and a classification of requirements, being of inherent or resistance type, and agents, being of degradation or/and rupture type, respectively, has been suggested. Service life vs. performance requirements has been discussed and a common principle for the criterion of the performance requirement, based on the instantaneous performance characteristic, has been put forward.

Further concepts to discuss and approximate may include, for instance, damage functions, dose-response functions, performance tests and accelerated exposure. Other examples could certainly be found within the domain of mechanical testing.

8 References

European Organisation for Technical Approvals (1997) *Assessment of working life of Products, PT 3 – Durability, Final Draft – March 1997*. EOTA, Brussels. TB 97/24/9.3.1.

International Organisation for Standardisation (1984) *Performance Standards in Buildings – Principles for Their Preparation and Factors to be Considered*. ISO, Geneva. ISO 6241-1984 (E).

International Organisation for Standardisation (1998) *Buildings – Service Life Planning – Part 1: General Principles*. ISO, Geneva. ISO/DIS 15686-1.

International Organisation for Standardisation (1999) *Buildings – Service Life Planning – Part 2: Service Life Prediction Principles*. ISO, Geneva. ISO/DIS 15686-2.

Lounis, Z., Vanier, D.J. and Lacasse, M.A. (1998) A discrete stochastic model for performance prediction of roofing systems, in *Construction and the Environment, CIB World Building Congress 1998, Gävle, Sweden, 7-12 June, Symposium A: Materials and Technologies for Sustainable Construction*, Vol. 1, pp. 305-13.

Lounis, Z., Lacasse, M.A., Siemes, A.J.M. and Moser, K. (1998) Further steps towards a quantitative approach to durability design, in *Construction and the Environment, CIB World Building Congress 1998, Gävle, Sweden, 7-12 June, Symposium A: Materials and Technologies for Sustainable Construction*, Vol. 1, pp. 315-24.

Masters, L.W. and Brandt, E. (1989) Systematic methodology for service life prediction of building materials and components. *Materials and Structures*, Vol. 22, No. 131, pp. 385-92.

The Council of the European Communities (1988) *Council directive of 21 December 1998 on the approximation of laws, regulations and administrative provisions of the Member States relating to construction products*. The Council of the European Communities, Brussels. 89/106/EEC.

133 INVESTIGATIONS OF MASS AND ENERGY FLOW IN EXISTING BUILDINGS

H. Kloft and J.D. Wörner

Darmstadt University of Technology, Germany

Abstract

The effects on the environment through the amounts of building material masses makes it necessary to direct and reduce the material and energy flow in the building sector. A problem arises from the lack of information regarding the material and energy movement especially in the area of building structures. In this paper investigations are presented regarding material and energy flows for existing residential buildings. At first residential buildings of varying ages but same type of construction were examined and compared. The results give information on the change of use of building materials in different age groups. Secondly present-day residential buildings with different type of construction were investigated. As results the diversity of influences of type of construction are shown. Finally the influence of service life and durability on the material and energy flows were examined. The shown investigations are to assist in closing the information gaps regarding material and energy flows for buildings.

Keywords: material and energy flow, existing building stock, primary energy contents, durability, service life

1 Introduction

The continual reduction and use of raw materials and energy is a basic prerequisition for the existence of today's industrial society. In nature, metabolisms are closed systems. In contrast, technical processes are characterised by a mainly linear course. Though in the past the re-use of materials from demolished buildings was taken for granted as the obtaining of new building materials was difficult and

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expensive, today ecological reasons force us to close the open process chains of the industrial production. In road construction, the flow of materials is already so developed that hardly any disposal site material is produced. In comparison, building construction is mainly characterised by open, i.e. non-recycling processes. In the future it will however not be sufficient to increase efforts in returning surplus building materials into the material circulation cycle. The complexity of effects on the environment through technical processes makes it at the same time necessary to reduce the material and energy input necessary for the construction and use of buildings.

The directing as well as the reducing of material and energy flow in the building sector is however encumbered with a number of problems. An important problem arises from the lack of information regarding the material movement especially in the area of building structures. This is mainly because material flow is only registered at the commencement in the production of the building materials, and at the end with the removal of the building waste. However, the movements taking place between production and disposal site are largely unknown. The knowledge concerning this is however an important assumption to not only develop strategies and planning instruments to direct and reduce the material and energy flows in building construction but also in order to fully evaluate buildings in the future with regard to their influence on their environment.

At the present time, two tendencies can be recognised in the assessment of material and energy flow in building construction. On the one side, detailed investigations at a micro-analytical level are carried out on individual building materials, on the other, macro-structures are determined such as the existing national German building stock for example. Incomplete in comparison are distinct statements regarding the material composition as well as the reciprocal interdependence and dimensions of the material and energy flows of individual buildings.

In the following, the results of investigations are presented regarding material and energy flows for residential buildings. The investigations were carried out at the Institut für Statik der Technischen Universität Darmstadt (Institute for Structural Analysis at the Darmstadt University of Technology) and were sponsored by the Fritz and Margot Faudi Foundation.

2 Material and energy contents of residential buildings constructed at different times

In order to return surplus building materials from building construction into the material circulation cycle, details regarding their quantitative amounts and their qualitative composition are of decisive importance. Working on the assumption of an average building structure life for residential buildings of 80 - 100 years, building rubble accumulated at the present time is made up of building materials which were produced at the beginning of the century. Within the framework of investigations carried out at the Institute for Structural Analysis on the "Usage of Building Materials and the Primary Energy Contents of Residential Buildings Constructed at Different

Times", 20 residential buildings of varying ages were examined with regard to their building material and mass composition. In addition, the primary energy contents of the building which are represented in the form of construction materials (PEI_H) were determined. This gives the accumulated primary energy expended in the production of the construction materials. The examined residential buildings represent typical solid-wall type construction, a distinction being made for single-family (EH) and multi-family (MH) housing.

The results show that the average building material masses for all examined buildings is 530 kg/m^3 gross cubic content (BRI); related to the relevant living areas this gives an average value of 2550 kg/m^2 living area (WFL). The examined single-family houses are on average some 15 - 25 % heavier than the multi-family houses. This is mainly due to the construction elements foundations, external walls and roofs. More than 80 % of the building material masses of the examined residential buildings can be apportioned to the loadbearing structure, less than 20 % to the finishes. For the buildings examined here the technical installation can be disregarded as far as mass is concerned. The investigations have also shown that more then 90 % of the building material masses of the examined housing of solid-wall type of construction are of mineral origin.

The graphic depiction of the used building materials for the investigated residential buildings related to the age of the buildings in figure 1 shows that the building material composition changes from one age group to the next. Whereas the concrete portion has increased since the mid 1920's, the portion of timber has decreased to under 5 %. Although the buildings of the age groups from 1918 to the present day are mainly examples of the solid-wall type of construction, the portion of block and brickwork walling decreases in this presentation. This illustrates the fact that the increased use of concrete is not only at the expense of the building material timber but also of block and brickwork walling.

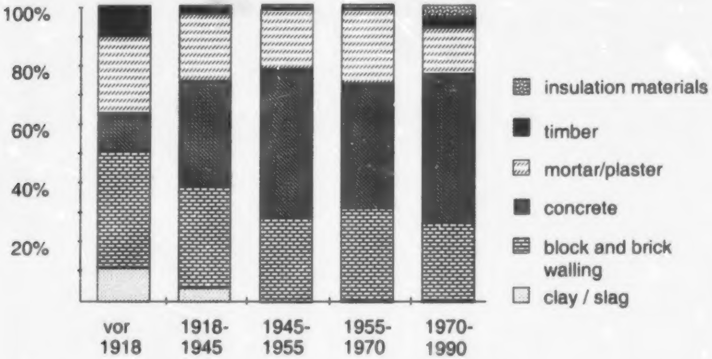

Fig. 1: Usage of building materials of various age groups (mass percentage)

The building materials clay and slag, which were still often used at the beginning of this century in timber beam ceiling constructions, have almost completely disappeared since World War II. The proportion of the collective group mortar/plaster/gypsum remains generally stable. For the examined residential buildings, the accumulated primary energy necessary to produce the used building materials (PEI_H) amounts to an average of approx. 900 MJ/m³ gross cubic content (BRI),equivalent to approx. 250 kWh/m³ gross cubic content (BRI). Here too the tendency can be seen that multi-family residential buildings considered in the absolute, show lower primary energy content analogous to the building material masses.

An interesting finding is depicted in figure 2 showing the relationship between primary energy contents and building material masses. The building material masses and primary energy contents of 5 multi-family houses of similar type of construction and cubic content of different ages are shown. It can be seen that newer buildings with values of around 2 MJ/kg have a higher PEI_H/building material masses level than older buildings (1.6 - 1.8 MJ/kg), the absolute values for the primary energy contents however can be lower than for buildings of earlier building age groups.

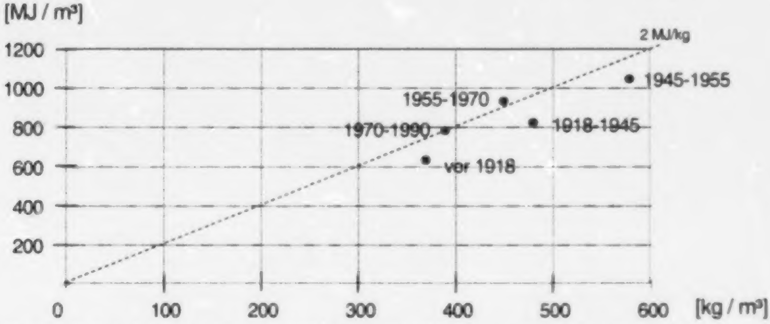

Fig. 2: Related total values of building material mass and primary energy input for five multi-family houses of various age groups

3 Material and energy contents of residential buildings built to present-day standards

The aim of the investigation of residential buildings of present-day standards is to show the diversity of influences of type of construction, choice of material, cubic content and other special conditions on the material and energy contents. For this, 15 residential buildings of differing types of construction and geometrical proportions but with approximately comparative heating energy values (in accordance with WSVO 95, the German Thermal Protection Ordinance of 1995) were examined. For the investigated buildings, the mean value for the building material masses amounted to 425 kg/m³ gross cubic content (BRI), and 866 MJ/m³ gross cubic content (BRI) for the primary energy contents, equivalent to 240 kWh/m³ gross cubic content (BRI).

Noticeable deviations from the mean value for the building material masses are apparent for buildings of timber-frame construction which are much lighter, and also for some solid-wall buildings which have undergone appreciable mass addition due to special building measures. Here one can note the influences of cellars or basement parking as well as marked enlargements to the building envelope in the form of dormer windows, bay windows, etc. Similar deviations to those for the building material masses can be seen for the primary energy contents. Here too, this indicates that heavy buildings have higher primary energy contents and lighter buildings are more favourable in this context.

Analogous to earlier buildings, it is apparent that even in present-day construction of solid-wall residential buildings, in the distribution of the building material masses on the loadbearing construction and finishes, some 80 % of the building material masses can be apportioned to the loadbearing structure. In contrast, clear differences are seen in the percentage distribution of the primary energy contents on the loadbearing construction and finishes. The results show that in buildings constructed to present-day standards there is an appreciable increase of the energy contents in the materials used for the finishing work. Whereas some 20 - 30 % of the energy content applied was necessary for finishing work for earlier building groups, these values vary today between 30 and 60 %.

Figure 3 shows the depiction of the total values of building material masses and primary energy contents related to the gross cubic content in a co-ordinate system.

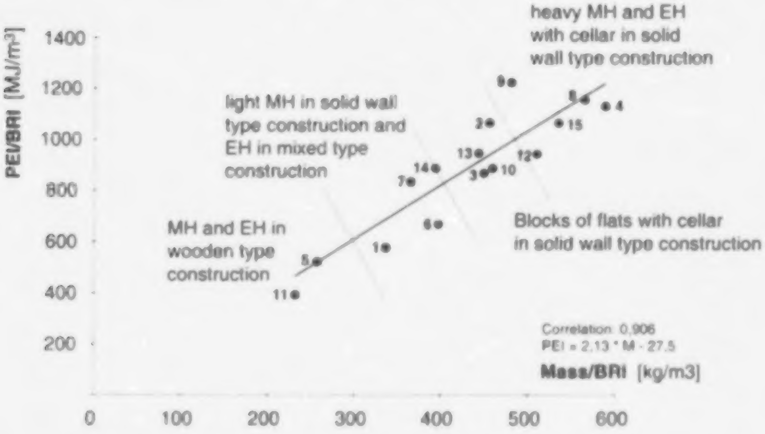

Fig. 3: Related total values of building material mass and primary energy input for fifteen residential houses with comparative heating energy values

The depiction of building material masses and primary energy contents reflects the constructional range of present-day buildings by a great mean variation of the absolute values. It is nevertheless noticeable that, despite the great mean variation of the absolute values, there is little mean variation for the PEI_H/building material masses level. With a correlation of 0.91, the examined buildings show a PEI_H/building material masses level of 2.1 MJ/kg. An exact observation of the individual construction groups shown in figure 4 represents that the loadbearing construction of the investigated buildings gives a very good linear correlation of 0.92 to a straight line with a gradient (PEI_H/building material mass level) of 1.47 MJ/kg.

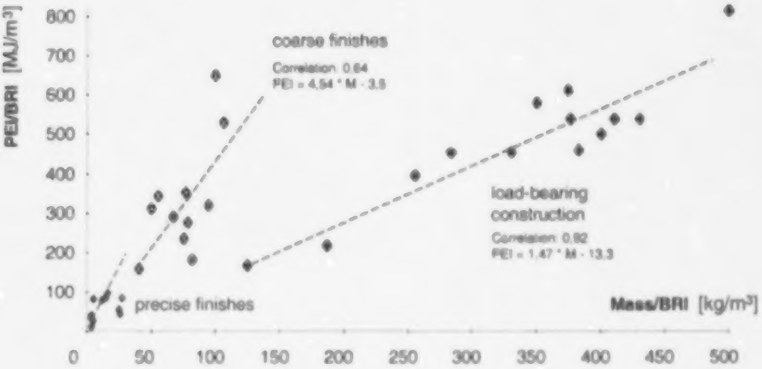

Fig. 4: Related total values of building material mass and primary energy input for the individual construction groups of fifteen residential houses

The large dispersion of values for the finishing materials is reflected here by the plainly perceptible reduction correlation of 0.64 for the coarse finishes and 0.44 for the fine finishes. This shows that the type and design of the loadbearing construction is of great influence on the absolute values of the building material masses and energy content of buildings, whereas the PEI_H/building material mass level of buildings is mainly determined by the materials used in the finishing work. The influence of the finishing materials increases as the loadbearing construction decreases in weight.

4 Material and energy expenditure during building use

In the following, the influence of the period of use on the material and energy flows in residential buildings are shown dependent on the energy standards and taking into consideration the technical and functional life span of the used materials. The investigation is based on three single-family houses of comparative geometric condition (cubic content, living area), which are however distinguished by having completely different energy concepts. A house built in accordance with the German

Thermal Protection Ordinance (WSVO-house), a low-energy house and a so-called passive-solar house have been used as examples.

The materials and energy contents at the time of construction (t = 0) in figure 5 show that for the WSVO-house (house built in accordance with the German Thermal Protection Ordinance) and the passive-solar house, the PEI_H/building material mass level has risen markedly. This seems understandable for the passive house, but seems surprising for the house designed in accordance with the thermal protection ordinance. The main reason for the rather high PEI_H/building material mass level of 2.59 MJ/kg for the WSVO-house is to be found in the peculiarity of the building construction: This building is actually a representative of a mixed-type of construction, with its solid external walls and wooden beam intermediate floor structure, but the lack of cellar and the otherwise solid floor slab which normally give a favourable PEI_H/building material mass level, here produce an increase of the total energy level.

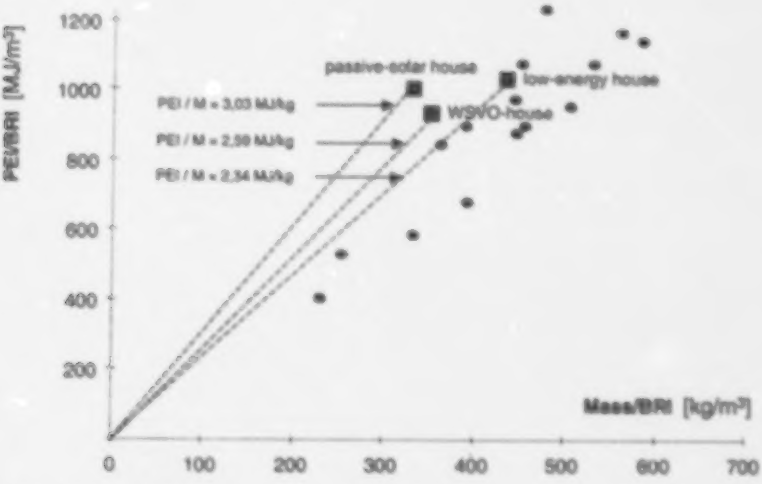

Fig. 5: Building material mass and primary energy input for three single-family houses with different energy concepts compared with the values of fig. 3

If one regards the accumulated building material masses over a period of 80 years, taking into consideration the technical and functional duration of use of the used materials, it is shown in figure 6 that the building material masses at the time of their construction are dominated by the loadbearing construction building elements, and with increasing time span of use, the finishing work materials increase in importance. With the examined passive-solar house, for example, the maintenance measures led to a doubling of the building material masses for the finishing works after a period of 80 years. All in all, however, no serious differences are shown with regard to the development of the material flows during the period of use, based on those of the differing energetic standards of the three single-family houses. The reason for this is surely that the improvement of the energetic standards are primarily concentrated on the building materials of the finishing works, and these materials do not produce a marked increase of the building material mass even with a doubling of their volume due to their low material density.

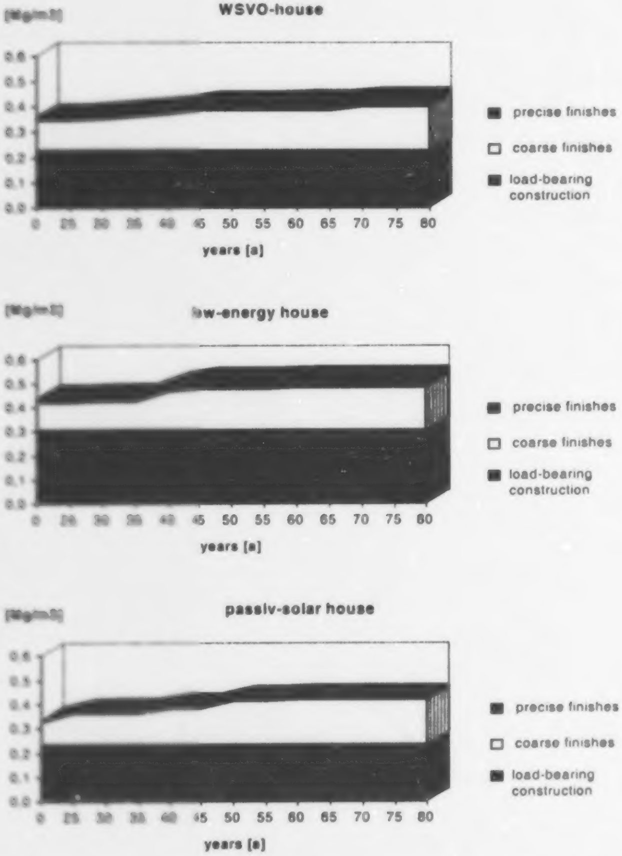

Fig. 6: Cumulative building material masses of three single-family houses with different energy concepts over a period of use of 80 years

This is, however, not the case regarding the accumulated energy expenditure. In the case of the house built in accordance with the German Thermal Protection Ordinance (WSVO-house) it is shown in figure 7 that the primary energy consumption for heating and the production of hot water as well as for household electricity considered over a duration of use of 80 years amounts to almost eighteen times the energetic input that is required for the production and maintenance of the building materials. Even though the primary energy expenditure for the production of the building materials used for the passive-solar house is almost double that needed for the building constructed in accordance with the thermal protection ordinance, the WSVO-house has a five times higher energy consumption in the total assessment after 80 years compared to the passive-solar house.

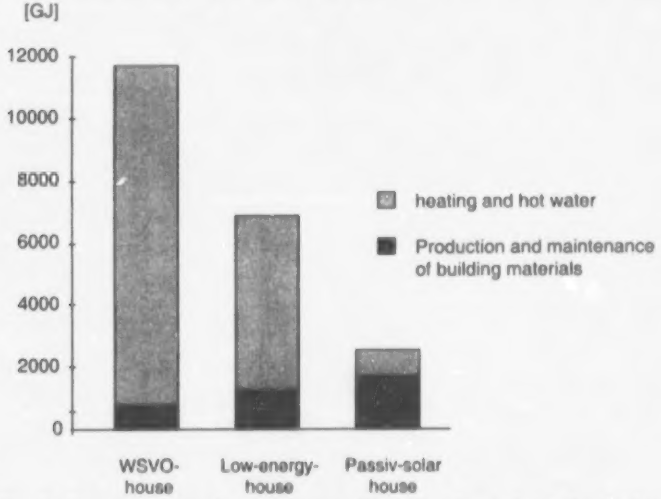

Fig. 7: Cumulative total energy expenditure for three single-family houses with different energy concepts over a period of use of 80 years

5 Prospects

The shown investigations are to assist in closing the information gaps regarding material and energy flows for buildings. A precondition for the application of a recycling industry in building construction is the consequent handling in the future of the flood of information and data on the total life span of buildings. For this purpose it is necessary to set up an „information cycle in building construction". This information cycle represents the linking up of all information relevant to the quantification of the material and energy flows of buildings, commencing with the production of building materials to the individual phases in the life of a building. The permanent focal point of the information flow is a Central Data Bank. Here all data are to be stored, updated and made readily retrievable. Each building phase in turn is represented by its own information cycle, which can be split into subordinate modules as required. Important for each further dividing up of the building phases is the maintenance of the information cycle as well as the linking together of all modules to the Central Data Bank. The data flow must not be only in one direction but there must be a permanent exchange of data and information during the individual building phases with the Central Data Bank.

At the present time a computer-assisted working instrument is being developed by the Institute for Structural Analysis at the Darmstadt University of Technology to record the material and energy application in the design of new buildings. A data bank model will be designed based on the knowledge gained in the carried out investigations which will permit a computer-aided compilation of the material and energy contents of buildings. The aim is to document the material end energy contents of buildings at the time of their construction with the assistance of the developed program, and to continue this later when alteration and renovation measures are carried out. A further aim is to apply the developed program as a design instrument already at the design phase for the conception of resource-optimised buildings.

6 References

Kloft, H. et al. (1996) An Investigation of the Use of Building Materials in Residential Premises Constructed at Different Times. *Congress Report 15th IABSE-Congress*, Copenhagen, pp. 481–86,

Kloft, H. and Wörner, J.-D. (1998) Investigations of Energy and Mass Flow in the Life Cycle of Buildings. *Structural Concrete 1994-1998 German Group of FIP to XIIIth FIP Congress*, Amsterdam, pp. 150-53

Kloft, H. and Wörner, J.-D. (1998) Investigations of Mass Flow in the Existing Building Stock. *IABSE-Colloquium Saving Buildings in Central and Eastern Europe*, Berlin, pp. 20-21

Kloft, H. and Wörner, J.-D. (1998) Investigations of Mass and Energy Flow in the Existing Building Stock. *Congress Report CIB World Building Congress*, Gävle, pp. 761-68

Kloft, H. (1998) Investigations of Mass and Energy Flow in residential buildings. *Dissertation at Darmstadt University of Technology*

Wörner, J.-D. and Kloft H. (1996) Untersuchungen zum Baustoffeinsatz und zum konstruktiven Aufbau von Wohngebäuden verschiedenen Baualters. *Congress Report International Symposium of CIB W67*, Vienna, pp.335– 40

134 DEGRADATION ANALYSIS BY STATISTICAL METHODS

B. Leira, J. Lingård, A. Nesje, E. Sund, and S. Sægrov
SINTEF Civil and Environmental Engineering, Trondheim, Norway

Abstract

Several utilities experience a great future challenge due to deterioration of properties, this being both buildings and infrastructure. As service life ends, there will be an increasing need of rehabilitation (i.e. renewal and maintenance). Most maintenance decisions up to now have been based on the so-called fire brigade strategy, i.e. to make spot repairs after the failure has occurred, or based on rules of thumb. In order to enhance maintenance and rehabilitation decision-making, it is essential to improve our understandings about the deterioration processes. A set of tools should be developed for decision support. These should be based on, or take into account, existing knowledge of failures. Statistical methods can be regarded as a way to organise this knowledge. This paper describes how statistical methods can be applied for forecasting rehabilitation needs. Examples of trend plots, survival methods, condition class transition and stochastic model parameter analyses from concrete structures, roads and water networks are shown. It should be emphasized that there are similarities in the way various constructions could be analysed.

Keywords: Built Environment, Construction degradation, Infrastructure, Life Cycle Management. Maintenance, Rehabilitation, Service life, Statistical methods

1 Introduction

Service life management is a research field where a broad spectrum of knowledge is needed to achieve results. A project called *Service Life Management of Buildings and Infrastructure* started at SINTEF in 1996, where one objective was to study how to use statistical approach in modelling degradation processes.

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A project team of researchers from sectors as highway engineering, water and wastewater, material and building, safety and reliability was established.

The term *service life management* is often used in conjunction with *durability or sustainability*. The consequence of limited durability is a need for maintenance and rehabilitation. The term *rehabilitation* here comprises different efforts to prolong the service life and to renew constructions.

There is a great need for tools for estimating service life or durability for materials, buildings and infrastructure. A major limitation is the lack of good quality data on construction failures, and methods to organise the information that really exists. Statistical methods may give contributions to decision support systems on maintenance and rehabilitation of buildings and infrastructure.

Independently of the kind of structure, it seems there are some similarities in the way constructions deteriorate. This gives reason to believe that the methods applied for studying the performance of one kind of structure could also be used for other structures. In general, reliable field- and laboratory measurement data are needed for the development of models for service life prediction. It might be that experiences from measuring and data collection could be transferred from one sector to another. Similarly, it seems reasonable that there could be a benefit from collaborative use of statistical methods between different building and infrastructure disciplines.

This paper highlights the use of different statistical methods, illustrated by practical examples form water network, highways and concrete structures. Some of the methods are of "easy- to -use"- category, and hence not based on complex theory or a lot of data (example: trend plots). The use of other methods requires understanding of advanced theory and computer software.

2 Application of different approaches

2.1 Trend plots and survival methods

2.1.1 Problems and objectives

The deterioration of water mains causes problems such as reduction in carrying capacity of pipes due to tuberculation, and bursts and leaks occurring at increasing rates along the pipe segments. The maintenance of water mains is thus increasingly demanded. In order to enhance maintenance decision making, it is essential to improve our understanding about the deterioration process of water mains and to develop appropriate predictive models that can assist us in making maintenance decisions.

The main objectives of a water network record analyses should include:

- Analyses of pipe inventory data and historical failure data registered in pipe network databases.
- Apply statistical models for forecasting future rehabilitation needs.

2.1.2 Application of statistical models

Two classes of statistical models could be distinguished, namely counting process models and lifetime models. With lifetime models, one can estimate the probability that a pipe will fail within a time horizon. With counting process models one can see the deteriorating (or improving) trend in time of a group of pipes and the rate of occurrence of failure (ROCOF) of a group of pipes.

Figure 1 illustrates the use of counting process models for the municipal water network in Trondheim. Systematic failure registration started in 1987, when a new water network database (Gemini VA) was available. The network has been divided into 10 classes (table 1), and the accumulated failure per observed length of the corresponding network has been plotted. It can be seen that the "class III", which corresponds to ductile iron pipes with a diameter 100-200 mm and installed between 1963 and 1975 has by far the highest failure frequency. The curve is nearly linear, which shows that this network class, although with a very high failure frequency, has not yet reached the state of "burning out".

Figure 2 shows a lifetime forecasting model, exemplified by the Cox Proportional Function for the Trondheim water network. From this figure, it can be read that 40 % of the ductile iron pipe network, which have been installed between 1963 and 1975, will have to be repaired or renewed within 20 years.

Table 1: Division of pipe network into condition classes
(Lei and Sægrov 1998)

Class	Installation year	Material	Diameter [mm]	# of pipes	# of failures
I	[1862, 1963]	Cast iron	<= 250	2449	532
II	[1862, 1963]	Cast iron	> 250	229	47
III	(1963, 1975]	Ductile iron	<= 250	2250	1314
IV	(1963, 1975]	Ductile iron	> 250	139	19
V	(1975, 1997]	Ductile iron	<= 250	690	56
VI	(1975, 1997]	Ductile iron	> 250	80	2
VII	(1975, 1997]	Ductile iron + Protection	<= 250	1160	28
VIII	(1975, 1997]	Ductile iron + Protection	> 250	74	0
IX	(1975, 1997]	Plastic	<= 250	686	9
X	(1975, 1997]	Plastic	> 250	49	0
The rest				646	263
Total				8452	2270

2.1.3 Application for other sectors

The counting process is being used within several building sectors. The Nelson Aalen plot represents the integration of the famous "bath tub curve", which illustrates the three major stages in a buildings life: the "burn in", the "operation" and the "wear out" periods. The plot is a robust method to show the current life stage of a construction or a building.

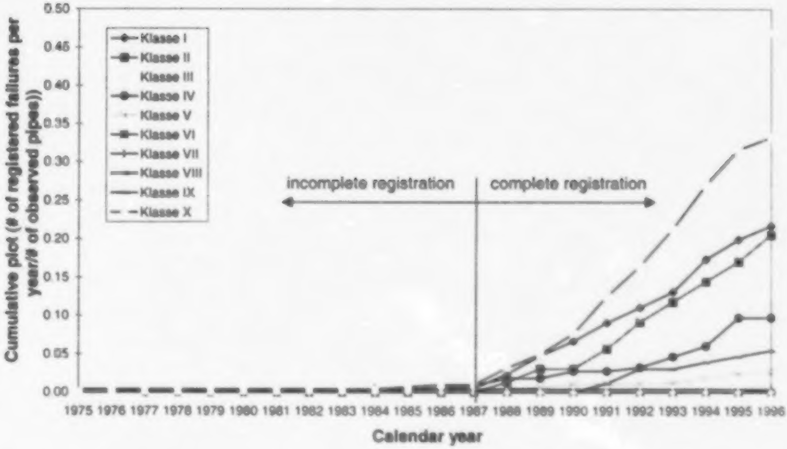

Fig. 1: Application of counting process in Trondheim water network (The term “klasse” refers to “class”

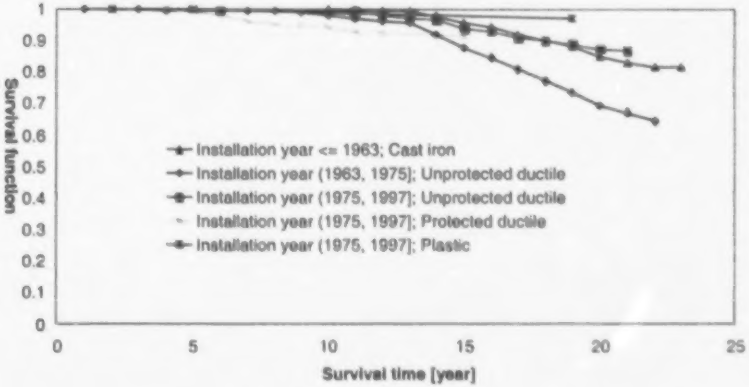

Fig. 2: Application of forecasting model (Cox Proportional Hazard Model) for Trondheim water network (Lei and Sægrov 1997)

2.2 Modelling condition deterioration by the use of Markov Chains

2.2.1 Introduction

One type of statistical modeling is the use of Markov chains. It has been used in several fields, e.g. road pavements and bridges to model how condition changes with time based on empirical data. These models can be used as a basis for optimizing maintenance and rehabilitation (M&R) strategies. To develop the models for effective use of the method a substantial amount of good quality data is needed. The Markov chain model used to model the deterioration of the pavement of a road network is used as an example throughout this section.

2.2.2 Method

First the appropriate condition parameters should be identified. These should be limited to the ones that are critical with regard to M&R being initiated. The value range of each condition parameter is subdivided into two or more condition intervals, thus forming discrete condition states. For road pavements, appropriate condition parameters could be: Rutting, roughness and surface distress. If the value range for each of these is subdivided into 4 condition states, then each pavement section in the network could be classified in one of 4x4x4=64 condition states.

The probability that a pavement in a specific class will degrade to another state when going from time *t* to *t+1* (the period usually being one year) is defined as the *transition probability*. By using empirical data for the existing road network, transition probability matrices are developed, e.g. using regression analysis. The road network can be divided into sub-networks depending on e.g. traffic volume, geografic regions etc., with transition probability matrices being developed for each sub-network. The effect of various M/R-operations is also modelled using transition probability matrices. The transition probability matrixes for each condition parameter have the following structure:

Table 3: Example of subdivision in condition states

Roughness:	1				2				3				4			
Surface distress:	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4
Rutting: 1	**111**	**121**	**131**	**141**	**211**	**221**	**231**	**241**	**311**	**321**	**331**	**341**	**411**	**421**	**431**	**441**
2	**112**	**122**	**132**	**142**	**212**	**222**	**232**	**242**	**312**	**322**	**332**	**342**	**412**	**422**	**432**	**442**
3	**113**	**123**	**133**	**143**	**213**	**223**	**233**	**243**	**313**	**323**	**333**	**343**	**413**	**423**	**433**	**443**
4	**114**	**124**	**134**	**144**	**214**	**224**	**234**	**244**	**314**	**324**	**334**	**344**	**414**	**424**	**434**	**444**

Year		t + 1			
	i/j	1	2	3	4
T	1	p_{11}	p_{12}	p_{13}	p_{14}
	2	p_{21}	p_{22}	p_{23}	p_{24}
	3	p_{31}	p_{32}	p_{33}	p_{34}
	4	p_{41}	p_{42}	p_{43}	p_{44}

Where t = year t
t+1 = the following year
1,2,3,4 = condition state number
i = condition state in year t
j = condition state in year t+1
p_{ij} = probability of transition from condition state i to j in the time period from year t to year t+1

Fig. 3: Principle of transition probability

The probability p_{11} is the probability that road sections in state one in year t will still be in the same state in year t+1. When one assumes that no road section can improve its condition without M/R-action then all the values below the diagonal in the matrix are zero (p_{21}, p_{32} etc.). A transition probability matrix for a M/R-action would on the other hand typically only have positive values below the diagonal.

2.2.3 Application of method

The method can be used to model the deterioration of e.g. a road network. These models can be used as an integral part of a network level management system, where the models are combined with methods for economic optimization of total costs for society (i.e. both road user and agency costs). There are plans to carry out a full-scale test of an existing Finnish system (Talvitie and Olsonen 1988) with Norwegian data during the winter 1998/99. A similar test has been carried out in Sweden each year from 1995 (Äijö and Tikka 1995). Using linear programming optimizing techniques the optimal distribution of the road network in the different condition states are found taking into consideration both the costs for M/R-actions and the road user costs. This is the steady state where the total costs for society are at a minimum. This is called the "long-term" analysis. In the following "short-term" analysis a M/R-program is found to minimize the gap between the current condition and the long-term optimal condition, taking into consideration e.g. budget constraints. This is illustrated in the figure below.

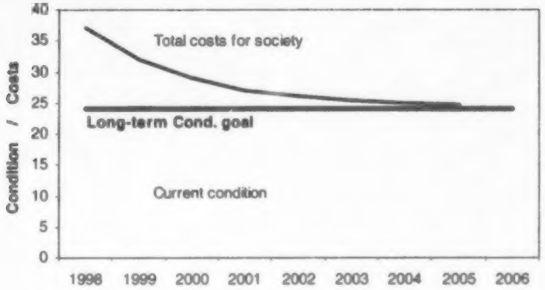

Fig. 4: Principle of short-term analysis

By analysing different budget-scenarios one can illustrate effects on condition and user costs to decision-makers. Other applications are to use the models in allocation of funds between regions and to identify M/R-strategies that will bring the network condition closer to the long-term optimal state.

2 2.4 Potential for using method in other sectors

The use of Markov chains to model condition deterioration can in principle be applied to any kind of infrastructure as long as the necessary data are available to generate the transition probability matrices with a satisfactory degree of accuracy. Markov chain modeling has been utilized in e.g. bridge management systems (PONTIS). Similar techniques have been used in modelling degradation of waste water systems (Aqua WertMin, Aqua Ingenieure GmbH), where the condition of sewers are classified from TV-inspection.

2.2.5 Data needs and quality

To develop the transition probability matrices used to describe the degradation in a probabilistic way it is a definite advantage to have a database with empirical data for the condition parameters that are being modelled. For testing the method the specific values in the matrices can be assumed, and refined as more and better quality data become available.

2.3 Analyses of chloride ingress in concrete structures

2.3.1 Introduction

Service life of concrete subjected to chloride ingress is typically predicted based on a deterministic approach. The values of relevant key parameters are estimated, and subsequently a deterministic formula based on a mechanical model is employed. This results in a point estimate of the service lifetime.

However, a number of the input parameters are to a higher or lesser degree subjected to uncertainty. Accordingly, it is more realistic to introduce a probability distribution for the service lifetime than a single point estimate. In the following, a procedure for probabilistic analysis is outlined. The resulting output from this type of analysis is also illustrated by a simple example.

The output from the analysis related to lifetime prediction is the probability distribution for the time until the chloride concentration at a given point has reached the critical threshold. Alternatively, the probability distribution of the chloride concentration at the given location for a fixed point in time can be employed

2.3.2 Lifetime prediction: methodology

The diffusion coefficient obviously plays a central role in relation to the speed of chloride ingress. The mathematical modelling of the diffusion process itself is generally based on Fick's 2^{nd} law. For a constant-in-time diffusion coefficient (in the following designated by D_0), the solution of this equation is readily expressed in terms of the complementary error function (erfc) as:

$$C(x,t) = C_i + (C_{sa}-C_i)\mathrm{erfc}(x/\mathrm{sqrt}(4(t-t_0)D_0)) \qquad (1)$$

- x - distance from the surface of the concrete to the location of interest.
- C_i – initial chloride concentration in the concrete (throughout the thickness)
- C_{sa} – chloride concentration of the external medium at the concrete surface
- C_{cr} – critical chloride-content of the concrete (at the reinforcement)
- D_0 – diffusion coeffisient
- t_0 – time to application of external chloride-containing medium

Probabilistic modelling of the diffusion process is based on the representation of the most uncertain parameters as stochastic variables. These may comprise the quantities C_i, C_{cr}, C_{sa} and D_0 introduced above. A set of representative statistical characteristics for these parameters are given e.g. in Karlsson and Poulsen (1995). A simplified probabilistic modelling for the purpose of a "preliminary" gross reliability assessment can be expressed by:

$$D_0 = D_{0Ref} * X_m \qquad (2)$$

where X_m is a model uncertainty factor which represents the total effect of the basic random variables. As an ad hoc representation, this variable is here taken to be Gaussian with expectation value 1.0 and C.O.V. 10% for the purpose of illustration.

The relevant probability to be evaluated in the present context, is the probability of "failure" at a given time t. The failure event corresponds to the chloride concentration at the location of the steel reinforcement being higher than a critical value. The critical level can also be modelled as a random variable as explained above. Numerical methods are employed for evaluation of this probability.

2.3.3 Illustrative example

Calculations are performed for a simplified example in order to illustrate the methodology and numerical procedures. For the first example, all input quantities are modelled as deterministic, except the diffusion coefficient which is represented according to the simplified model based on a single uncertainty factor which was described above.

The values of the initial chloride content, the surface concentration and the critical concentration are taken to be: $C_i = 0$, $C_{sa} = 0.4\%$ and $C_{cr} = 0.1\%$. The time of exposure is set equal to $t_0=0$, and the thickness of the concrete reinforcement cover is $x_{reinforcement} = 4$cm. Two radically different values of the "reference" diffusion coefficient are employed, i.e. $D_{0Ref}=290(\mathrm{mm}^2)$/year and $D_{0Ref}=10(\mathrm{mm}^2)$/year, respectively. The resulting probabilities of failure for the latter case are shown in Figure 5 for different service times.

It is found that a failure probability level of 10^{-1} is reached after roughly 53 years for the lowest diffusion coefficient.

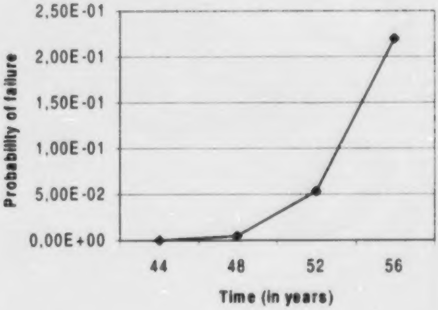

Fig. 5: Failure probability

2.3.4 Data collection

For a given concrete batch or for a given concrete structure, the chloride diffusion coefficient is frequently estimated by performing laboratory tests for a limited number of sample specimens. Based on the outcome of such tests, improved prediction of service lifetime can be performed. The results from the tests can be incorporated in the statistical modelling by improving the corresponding uncertainty measures.

Several test methods exist. In the following, we consider three different methods:

(i) Chloride migration
(ii) Submerged chloride ingress
(iii) Chloride spraying

By comparing results obtained by application of the different procedures to the same test specimens, systematic assessment of each procedure can be achieved. This also provides the basis for regression analysis and statistical modelling that can be applied also in the absence of test results. An *a priori* statistical model can then be established. An example of results obtained by processing of data from a comprehensive data base is provided by Figure 6.

The results presented in this figure is obtained from submerged chloride ingress tests. The mass ratio on the horizontal axis accounts for combined mass of cement and silica (denoted by s in the figure) whenever the latter is present. From the figure, it seems that a bi-linear regression curve with a discontinuous slope at a mass ratio of roughly 0.4 might fit the data. However, a significant scatter of the data is also noted, which clearly illustrates the relevance of statistical modelling for this key parameter.

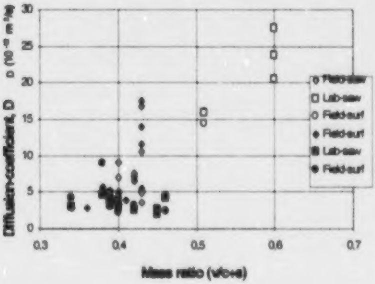

Fig. 6: Diffusion-coefficient test data obtained for submerged chloride migration (Lindgård, 1997)

2.3.5 Potential use of method in related areas

The present analysis approach can be applied to similar degradation phenomena for which a deterministic/mechanical model exist, and where parameter uncertainties are present. An example is corrosion of steel components such as pipes.

Time-dependent phenomena such as creep and relaxation can also be incorporated. However, in some cases a relatively involved statistical modelling of the loading process will typically be required.

Utilization of numerical procedures (e.g. stochastic finite element methods) for solution of more realistic time- and space-dependent parameters must be addressed, Engelund et.al. 1995.

3 Conclusions

Statistical methods can be used for decision support on maintenance of buildings and infrastructure. It has been shown that similar methods can be applied for different kinds of constructions. Methods based on statistics, however, demand long-term and good-quality data. Statistical methods will in itself be a tool for better understanding of physical deterioration processes and should be used supplementary to the knowledge and experiences of materials. Table 3 below gives a survey of some actual statistical methods and their application.

Table 3: Application of different statistical methods for degradation analyses

Class	Methods	Sector/ case	Data needs:
Trend plots	Nelson Aalen plots	Water networks, highways, bridges, dams, quays, building elements	Longterm data registration
Survival methods	Poisson, Weibull, Cox, Herz distributions	Water mains	Longterm data registration
Condition class transformance	Markov chains	Highways and roads, pipe networks, dams, bridges, quays	Large data volume
Stochastic analyses of parameters in deterministic models	Structural Reliability Methods (SRM) Limit State Design (LSD)	Structural systems and components	Test data (in-situ and laboratory)

4 Acknowledgement

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5 References

Aqua Ingenieure GmbH: AquaWertMin: Programmdokumentation. Saarbrücken, Germany (in German)

Engelund,S. ; Sørensen, J.D. and Krenk, S: "Estimation of the Time to Initiation of Corrosion in Existing Uncracked Concrete Structures", Applications of Statistics and Probability, Eds. Lemaire, Favre & Mebarki, Balkema, Rotterdam 1995, ISBN 90 5410 563 1.

Karlsson, M. and Poulsen, E. (1995) "Design of Rebar Concrete Covers in Marine Concrete Structures – Probabilistic Approach", RILEM Workshop, St-Remy-les-Chevreuse, France.

Lei J and Sægrov S (1998) Statistical Approach for Describing Lifetimes of Water Mains – Case Trondheim municipality, Norway. IAWQ 19th Biennal International Conference on Water Quality, Vancouver 21 – 26 June 1998.

Lindgård,J. (1997) "Gjennomgang av Laboratoriemetoder for Kloridinntrengning" (In Norwegian) , SINTEF Report, 1997-05-20.

Madsen,H.O. ; Krenk,S. and Lind, N.C. (1986) "Methods of Structural Safety", Prentice-Hall, Inc.; Englewood Cliffs, NewJersey.

Talvitie, A and Olsonen R (1988): Selecting Asphalt Concrete Condition States for Finlands Highways. Finland Highway and Waterway Adm., Paper presented at the 67th annual meeting of the Transportation research Board, Washington DC, Jan 1988.

Äijö J and Tikka K 1995: Översiktlig vägnettsanalys i Sverige, Test av system 100. Gen Oy 1995. Unpublished version.

135 ASSESSING THE REMAINING SERVICE LIFE OF EXISTING BUILDING COMPONENTS FOR INSURANCE

P.D. Mayer[1] and P. Wornell[2]

[1]Housing Association Property Mutual, London, UK, [2]Building Performance Group, London, UK

Abstract

Housing Association Property Mutual (HAPM) required a methodology to enable the remaining service life of existing components to be assessed for a new insurance product. A 35 year structural and non-structural defects cover is provided for existing housing stock comprising low rise, medium rise and high rise types, of both traditional and modern construction. A database structure was developed to allow systematic identification and appraisal of building components. Various service life assessment methods and models are considered to provide a framework to enhance site inspection objectivity and quantify future service lives to allocate an insurance life: a) visual condition assessment, b) modification of a reference service life using factors, c) scales of component condition, d) rates of deterioration, e) failure patterns, f) probabilistic models and g) service life forecasting guidance based on ISO/DIS 15686–1. Examples illustrate how the HAPM service life assessment model works in practice and is underpinned by theoretical analysis. The information gathering process and appraisal of service lives using these approaches could benefit by application of an expert computer system, incorporating fuzzy logic and rule induction software to assess uncertainty by computer generated rules.

Keywords: component database, component insurance, failure patterns, expert computer systems, prediction methods, remaining service life, Weibull distribution.

1 Introduction

Housing Association Property Mutual (HAPM) has nearly 10 years experience insuring new buildings and existing buildings which have been rehabilitated against the risks of major defects and premature component failure (Holmes and Wornell, 1994). A new insurance product — HAPM Dwellings — has been devised to cater for

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large scale transfer of housing stock from Local Authorities to Housing Associations. Such estate transfers required an insurance methodology which could provide 35 year structural and non-structural defects cover to existing components.

A methodology for assessing new components and allocating insurance lives has been established for HAPM's new build insurance (Bourke, 1996). Construction Audit Limited is a subsidiary of Building Performance Group (BPG) which provides the HAPM Technical Audit Unit (TAU). Both BPG and TAU are involved in component durability research. Their work comes together in publishing the Component Life Manual (Wornell *et al*, 1992) where shorter life components are distinguished and given a specific insurance life based on their expected service life.

2 Component database structure

HAPM insures housing against inherent defects which result in damage. A claim is made when a component is damaged i.e. fails. The component database is structured for insurance process and is based on components. Components encompass components (e.g. roof tiles), materials (e.g. insitu concrete floors) and assemblies (e.g. balustrades).

Structural and non–structural component distinctions as well as the consequences of failure are considered within the audit process (Holmes and Wornell, 1994).

2.1 New build component database

The new build insurance system includes a component database modeled on the Component Life Manual. Other component structures were considered such as the 'Construction indexing manual' (CI/SfB, 1991) but found to be inappropriate for the insurance context. The process of identifying components is hierarchical.

- *Elements*: e.g. foundations, flat roofs. In total there are 29 elements.
- *Component types*: the principal components comprising an element. The element flat roofs includes membrane coverings, decking and structural timbers.
- *Component sub–types*: a division of component types based on materials. Flat roof membranes include asphalt and double layer bitumen membranes.
- *Class category*: components are ranked by durability and assigned insurance lives. Best quality polymer based double layer bitumen is allocated a 20 year insurance life, an average quality glass–fibre based system is insured for 10 years.

2.2 Extension of the new build component database

The scope of the new build component database was found to be insufficient for stock transfers which include low rise, medium rise and high rise housing types, built in the 1900s to the present, using both traditional and modern construction methods. The database was enlarged by addition of some eighty component types with associated sub–types and classes. At the last count the component database covered over 400 component types, 1600 component sub–types and 9000 component classes.

2.3 Relationship between predicted service lives and 'insurance lives'

The insurance lives listed in the HAPM Component Life Manual are cautious estimates of durability based on predicted service lives which are modified to reflect the actuarial basis of premature failure. Durability is defined in terms of years a component is expected to function before failure i.e. a predicted service life (Moss, 1995). Premature failure occurs where a component requires complete replacement or extensive repairs within the HAPM insurance life i.e. before the predicted service life.

The research to enable precise adjustment of insured lives to provide estimates of service lives remains to be carried out (Bourke, 1994). Nevertheless, insured lives are typically targeted to reflect 80% of the predicted service life (Moss, 1995).

3 Auditing: component identification and insurance life allocation

Identification of components within the database requires 'stepping down' the hierarchy of element, component type, component sub–type and component class.

3.1 'Nominal' insurance lives at sub–type level

Clearly not all existing components can be identified visually at the most detailed level, i.e. by component class. An auditor may not be able to determine the grade of chipboard used for a ground floor decking but should be able to distinguish at sub–type level between plywood, orientated strand board and softwood boards.

The insurance audit system allows identification of existing components at sub–type level. To facilitate allocation of an insurance life a 'nominal' insurance life is associated with each component sub–type.

The 'nominal' insurance life concept is based on the insurance life of the most likely class of component used in the type of construction commonly encountered.

3.2 Concealed components

Typically concealed components include foundations, lintels, sleeper walls and wall ties. Concealed components may be critical when considering the longevity of a building, hence it is important they are assessed for insurance life allocation.

The auditor may have access to as–built drawings. An estate transfer may include a pilot scheme providing an opportunity to inspect concealed components.

Thorough inspection of the building should enable assessment of consequential defects associated with concealed components, for example, cracking of horizontal mortar courses may indicate wall tie corrosion. In which case further investigation would be requested. The auditor may go beyond a visual inspection by conducting wall–tie frequency detection tests at critical points such as gable verges. Detailed examination with a boroscope of wall cavities or floor voids should provide pointers to identify and assess the condition of components.

In the case of foundations there may be no circumstantial evidence to identify the type of construction or material used, the auditor may use a generic sub–type 'foundations'. Supposition is avoided, if an assumption is made about an existing component this is recorded in the insurance report.

3.3 Insurance life allocation

New components are automatically allocated the full insurance life associated with their component class by reference to the Component Life Manual.

Existing components are assigned a condition status based on site inspections. Component condition guides the allocation of insurance lives (Table 1).

Table 1: Component condition and insurance life allocation

Condition	Insurance life
As new	Full insurance life e.g. as component class or 'nominal' life.
Part worn	Less than 'nominal' life, in five year bands to a minimum of 5 years.
Needs replacement	No insurance life.
Concealed	A 'nominal' life, in five year bands subject to confirmation of there being no symptoms to indicate failure.

The condition of an existing component provides a guide to the rate of deterioration previously experienced. The auditor makes an assessment of future longevity based on quality of material, design, workmanship, external/internal environment and usage to assess future service life and then allocate an insurance life.

The process of allocating an insurance life for an existing component is qualitative yet is based on quantitative models. Theoretical and quantitative models are established in the literature, however, detailed guidance at component level for the complete range of components HAPM encounters is not readily available.

4 Models to forecast the remaining service life of components

Seven service life forecasting models or approaches have been identified to provide a working methodology for insurance purposes. They form a framework to enhance site inspection objectivity and a strategy to quantify future service lives to guide allocation of insurance lives.

4.1 Visual condition assessment

This is a qualitative approach, the remaining life of a building component is assessed by reference to its existing condition. It is a method commonly used for condition surveys. There are similarities between the processes used for insurance of existing components and the assessment of components for condition surveys. Components are considered within a definite time scale, 35 years or less. Time to replacement, and insurance lives are commonly expressed in five year time bands.

The maintenance regime is defined and accepted as a constant. In the insurance model a minimum maintenance regime is stipulated as a part of the life assignment.

The sampling strategy underlying condition surveys has application to auditing large scale stock transfers. The aim is to reduce risk by optimizing the trade off between sample frequency and accuracy (Mayer *et al*, 1994). In the insurance model a representative sample of all the components is inspected. The percentage will depend on the building type, construction and components encountered.

- *External components*: 100% of the exterior of a single block of flats would be assessed. A transfer of 50 similar two storey dwellings may have a two stage approach to sampling; the first stage involving an overview to identify and appraise typical or worst case dwellings, the second stage may involve a forensic external survey of one or more properties provided they are representative of the others.

- *Internal components:* Public areas, shared services and high risk components would generally have 100% assessment.
- *Internal components:* Private areas and services would be assessed in a similar manner as the two stage process for external components.

4.2 Modification of reference service life by factors

The factor method of estimating component service lives developed by the Architectural Institute of Japan, (1993) is incorporated in ISO/DIS 15686–1 — general principles of service life planning. Components are allocated a reference service life which is adjusted positively or negatively by multiplication of factors which represent, numerically, quality levels of a) materials and components, b) design, c) sitework/execution, d) indoor environment, e) outdoor environment, f) operating characteristics and g) maintenance level.

This method would readily adapt itself to an insurance model by factoring the reference service life to take into account the insurance risk. Knowledge of age and condition of an existing component would provide a control check on the assumptions of the factor model. The remaining service life could simply be calculated by subtracting the age of the component from the estimated service life. The insurance life would be a proportion of the remaining service life. While there are theoretical worked examples of the factor approach there is little direct experience of using the method. Consideration is also required of the effects of dosage variations and agents in combination. Research is being carried out to quantify factors and evaluate the mathematical relationships between factors (Hovde, 1998).

Nevertheless, the factor method provides a valuable framework within which to assess principal agents of deterioration and to adjust remaining service lives positively or negatively. In–house notes for auditors provide guidance on agents of deterioration, key failure modes and durability issues. In the context of allocating insurance lives to existing components the insurance life or 'nominal' life is increased or decreased in five year steps related to positive or negative factors.

4.3 Scales of component condition

A series of reference pictures, photographs or sketches is compiled for each component showing stages of deterioration, for differing modes of failure, from new to failed.

4.3.1 Advantages of this approach include:

- The actual condition can be compared to an objective standard.
- Auditors' assessments should be more consistent.
- Position on the condition scale and associated mode of failure clarifies the allocation of an insurance life.
- Stochastic modeling techniques are being developed to provide performance prediction taking into account uncertainty and variability associated with component quality, deterioration agents, quality of design, workmanship and maintenance regime (Lounis *et al*, 1998a).

4.3.2 Disadvantages:

- Information is not readily available, although academic papers suggest research has been carried out (Greer and Malek, 1998 and Brite-Euram 4213, 1996).
- Condition assessments on site would necessitate auditors carrying a potentially huge manual. Auditors could use photographs from site to compare with condition scales in the office, however this looses directness of observation.

Assessment by reference to condition scales remains an attractive method. Condition scales for commonly encountered components could be established and expanded as and when new components were encountered. Differing components constructed of the same material should follow similar failure patterns for given conditions. A set of condition scales for say, internal timbers would cover a large number of component sub–types (e.g. floor joists, wall plates, rafters and the like).

4.4 Rates of deterioration

The rate of component deterioration is a valuable piece of information to aid the process of service life assessment. The condition of an existing component at the time of inspection gives an indication of the rate of deterioration for a given set of conditions.

Three patterns of deterioration are recognized a) constant rate of deterioration — decomposition of timber, b) decreasing rate of deterioration — carbonation of concrete and c) increasing rate of deterioration — freeze–thaw action on concrete (Matsufuji *et al*, 1996). However, the pattern of deterioration is rarely sufficient in itself to complete the assessment of future service life.

In the case of a timber joist, repairs may be carried out which arrest decay and prevent further deterioration. The condition of the joist may not be as new but the remaining service life could equivalent to that for a new joist.

The pattern of concrete carbonation is only a guide to estimate the future life of a concrete structure The rate of carbonation, depth of reinforcement cover, quality of reinforcement, exposure and degree of pollution are other factors which would be considered. The carbonation process is influenced by three main variables: relative humidity, cement content and ambient CO_2 (Keršner *et al*, 1996). The implication for assessment of future service life is to concentrate on the dominant factors which influence the service life of a component within the framework of a particular pattern of deterioration.

While the action of free–thaw cycles may lead to accelerated deterioration of concrete, the frequency of freeze–thaw cycles as a climatic variable is critical in assessing the future service life of exposed concrete. Notwithstanding considerations of climatic change the general approach is to assume that future climatic conditions broadly follow the pattern of the recent past. By establishing the historic rate of deterioration the future rate and point of failure may be calculated.

To assess future service life a definition of component end–state or moment for replacement is required. This is taken from BS 7543 as the point at which excessive expenditure is required on operation, maintenance or repair. In insurance terms the quantification of component end–state is linked with performance of function, threat of damage as well as component deterioration.

4.5 Failure patterns

There are numerous statistical functions to model component failure; the Weibull distribution describes a variety of failure patterns (Bartlett and Simpson, 1998). Where possible results from published research are assessed to inform auditors judgement about the future service life of components for the purposes of allocating an insurance life. This section discusses three principal patterns of failure associated with components to enhance the assessment of future service life. Failure pattern models have been applied to the problem of maintenance optimisation (Moubray, 1997); the principles can be adapted to service life assessment.

4.5.1 Burn–in failure

The Weibull shape parameter is less than 1. Failures of this pattern can be discounted as we are dealing with existing buildings. Components which have failed due to burn–in failure will have been repaired or replaced.

4.5.2 Random or constant failure

Where the Weibull shape parameter is 1. While it is not possible to predict when any particular component will fail, the mean time between failures may be calculated. For purposes of assessing the residual life of components the mean time to failure can be treated as an average life. Components such as ventilation fans with bearings follow this type of failure pattern. The insurance life associated with these components takes into account the risk associated with random failure.

4.5.3 Age–related failure

This is the case when the Weibull shape parameter is greater than 1. Where the Weibull shape parameter is between 1 and 2 a pattern showing increasing probability of failure is described. Typically this pattern is associated with fatigue related failures, for example, a pump working over capacity. The rate of failure may vary considerably from weeks to decades. Where the rate of failure is very low this curve may approximate to that of random failure, with similar consequences for the service life of components.

A shape parameter of 3.2 or more tends towards a normal (bell–shaped) distribution. An assessment of survey data for the estimated service life of over 30 components showed that a significant number of components such as softwood windows displayed a normal distribution (Bourke, 1994). This information has been applied to the quantification of insurance lives which represent a time before the average service life.

In practice component failure patterns do not always conform to one of these Weibull distributions. The deterioration of masonry can be described by a three component composite model representing the cyclical mechanism of surface layer delamination due to free–thaw weathering (Molina *et al*, 1996).

A combination of random followed by age–related failure may be a more accurate description of the failure pattern experienced by external components such as roof tiles. In which case the onset of the age–related failure part of the curve would represent the service life.

Where failure patterns can be associated with components they provide a useful framework in which to judge their remaining service life and allocate a nominal insurance life.

4.6 Probabilistic models

The prediction of service lives has been considered as a probabilistic reliability problem (Lounis *et al*, 1998b). Guidance on the values to input into the theoretical equations would determine how these models could be applied in the context of remaining service life assessment.

4.7 Service life forecasting guidance — based on ISO/DIS 15686–1

The international standard gives guidance on service life planning in the context of imperfect knowledge. Two methods for forecasting the service life of buildings are proposed. The factor method has been commented on earlier.

Use of test data is the second method for predicting service lives. Where relevant test data is available the decision tree procedure as outlined in ISO/DIS 15686–1 and developed in ISO/CD 15686–2 is followed to provide an input to service life forecasting.

The office has built up a large data base of published service life estimates which is used in the evaluation of remaining service lives. In-house experience based on defect investigation and insurance claims provides feedback from practice.

Table 2: The process of allocating an insurance life.

Allocation of insurance lives — process	Guidance
1. *Preparation* — As built drawings, information from pilot schemes, condition surveys, specialist surveys or reports. Documentation for any proposed works.	Published information on defects/durability of similar buildings.
2. *Identification* — Identify components, ideally the manufacturer/product or by principal material as defined by component sub-type categories.	Classification based on the HAPM Component Life Manual.
3. *Determine condition* — visual inspection, mechanical probing/testing, opening up, boroscope investigation. Specialist reports e.g. drainage or concrete corrosion.	In-house notes on condition scales and inspection procedures.
4. *Construction* — Consider design aspects which influence durability. Check workmanship especially at junctions between different components and materials.	Inspection is facilitated with site visits during construction work.
5. *Agents of deterioration* — Consider agents of deterioration, modes of failure, rate of deterioration, external environment, internal environment, anticipated in-use conditions, effects of dosage variation and agents in combination.	In-house notes on principal agents and rates of deterioration, modes of failure and key durability issues.
6. *Remaining service life assessment* — Evaluation of likely remaining service life based on current condition and age of component in the context of the overall construction and agents of deterioration.	In–house notes on failure patterns, service lives based on published sources and experience.
7. *Insurance life allocation* — Determine the insurance life by reference to and comparison between the remaining service life and the nominal life.	Nominal insurance life dictates maximum insurance life.

5 Methodology to assess remaining service lives of components for insurance

The methodology used by HAPM auditors to assess the remaining service lives of components and the allocation of insurance lives is summarized in Table 2. At each stage of the process rules are provided. In practice the methodology draws on each of the service life assessment models described in this paper. The process is rarely linear as suggested by the table. Experience to date suggests the process is iterative; as more information becomes available and the work on site progresses a clearer picture of component type, condition and construction is gained. Feedback from in-house research and guidance about condition scales, failure patterns, rates of deterioration enhance the objectivity of site visits and inform assessment of remaining service life. The allocation of insurance lives is a function of the remaining service life.

Consistency is achieved by the insurance structure of maximum insurance lives and the principle of allocating insurance lives in five year bands. This restricts the range of choice so minor variations in the qualitative approach are limited. The office environment encourages informal discussions between auditors to compare and norm results. Formal meetings and joint site inspections are arranged to discuss particular difficulties. The quality control system within the office ensures that each judgement about insured lives is checked at each stage of the reporting process.

6 Expert computer systems

The use of knowledge–based expert systems for service life prediction has been suggested (Frohnsdorff and Martin, 1996). The HAPM model could benefit from application of expert systems. The condition of existing components and factors which influence future service life vary on a continuum. Attaching a quantitative value to continuous variables is problematic particularly where measures of condition or climate require specialist equipment. The mathematics of fuzzy logic enables non–specific variables to be modeled. The output expresses remaining service life as a probability. The key to using fuzzy logic is in defining rules to underpin the model.

A complementary approach would be to input the known data into a 'rule induction' or 'neural software' system to generate rules from patterns which may emerge from the data. The information and rules generated from these systems may be incorporated in a knowledge–based expert system which encapsulates the decision making process for remaining service life assessment and insurance life allocation.

7 Conclusion

The HAPM methodology for assessing remaining service lives of components for the purposes of allocating insurance lives is eclectic. It embraces aspects from each of the service life assessment models outlined above. The decision making approach is pragmatic — we do not have perfect knowledge but we are making the best judgement we can, informed by up–to–date research, methodologies and guidance.

8 References

Architectural Institute of Japan. (1993) *Principle guide for service life planning of buildings*, English edition.

Bartlett, E. V. and Simpson, S. (1998) "Durability and reliability, alternative approaches to assessment of component performance over time", in *Proceedings of CIB WBC, Gävle*. pp 35 – 42.

BS 7543. (1992) *British Standard guide to durability of buildings and building elements, products and components*. BSI, London.

Bourke, K.P. (1994) *Estimating building component durability for the purpose of risk management*. Unpublished M Phil thesis, University of the West of England.

Bourke, K.P. (1996) Model for insuring durability of building components, in *Proceedings of Dbmc7 Stockholm*, E & FN Spon, London, pp1223 – 1230.

Brite–Euram 4213. (1996) Condition assessment and maintenance strategies for buildings and building components. Brite–Euram project 4213 for CEC.

Frohnsdorff, G.J. and Martin, J.W. (1996) Towards prediction of building service life: the standards imperative, *Proceedings of Dbmc7 Stockholm*, pp1417 – 1428.

CI/SfB. (1991) *Construction indexing manual*. RIBA Publications, London.

Greer, W.C. Jr. and Malek, W. C. (1998) Information system for maintenance of 2500 buildings: a case history, in *Proceedings of CIB WBC, Gävle*. pp585 –592.

Holmes, R. and Wornell, P. (1994) Optimising defects control through design and insurance, in *Pro int. sym. dealing with defects in building, Varenna*, pp553–562.

Hovde, P. J. (1998) Evaluation of the factor method to estimate the service life of building components, in *Proceedings of CIB WBC, Gävle*, pp223 – 232.

ISO/DIS 15686–1. (1998), *Buildings - service life planning. Part 1 : General principles*, ISO/TC 59/SC3/WG9, "Design life of buildings" BSI, London.

ISO/CD 15686–2. (1998), *Buildings - service life planning. Part 2 : Service life prediction*, BSI, London.

Keršner, Z. *et al.* (1996) Service life prediction based on carbonation in *Proceedings of Dbmc7 Stockholm*, E & FN Spon, London, pp13 – 20.

Lounis, Z. *et al.* (1998a) A discrete stochastic model for performance prediction of roofing systems, in *Proceedings of CIB WBC, Gävle*, pp305 – 314.

Lounis, Z. *et al.* (1998b) Further steps towards a quantitative approach to durability design, in *Proceedings of CIB WBC, Gävle*, pp315 – 324.

Matsufuji, Y. *et al.* (1996) Service life predictive method of building materials, in *Proceedings of Dbmc7 Stockholm*, E & FN Spon, London, pp45 – 53.

Mayer, P. D. *et al.* (1994) *Stock condition surveys: a basic guide for housing associations*, National Federation of Housing Associations, London.

Molina, C. *et al.* (1996) A service life prediction model for masonry based on accelerated testing and Weibull interpreted testing results, in *Proceedings of Dbmc7 Stockholm*, E & FN Spon, London, pp75 – 85.

Moss, G. (1995) *Lifespans of building components*, HAPM Technical Note number 6 – June 1995, HAPM, London.

Moubray, J. (1997) *Reliability–centred maintenance*, Butterworth Heinemann, Oxford, United Kingdom.

Wornell, P. *et al.* (1992) *HAPM Component Life Manual*, E & FN Spon, London.

136 QUALITATIVE SIMULATION FOR ASSESSMENT OF CHEMICAL-PHYSICAL COMPATIBILITY AMONG BUILDING COMPONENTS

B. Naticchia

University of Ancona, Italy

Abstract

Behavior of building systems depends on a variety of both chemical and physical phenomena, someone typical of natural material decay, others connected to reciprocal negative influence between materials. These phenomena are frequently each other closely interrelated, therefore modeling processes in order to predict behavior of building components is often too hard to be pursued with numerical approaches. In fact great difficulty is encountered in joining many numerical models in order to simulate behavior of real systems taking contemporarily account of any relevant phenomena involved in decay processes. This is mainly due to changes of system structure induced by irreversible phenomena like properties alterations of materials, modification of components relationship produced by chemical reactions, so that the behavior is characterized by a remarkable discontinuity. Analogous difficulties, due to the complexity of causal structure of events and phenomena involved in component decay, are encountered in interpreting experimental results.

The method discussed in this paper, could be a contribute in dealing in a formal way with the issue of predicting chemical-physical incompatibility problems in innovative building systems in order to pinpoint correct strategy of improving system reliability. To this end we propose a model, based on 'qualitative simulation approach' capable to represent the dynamics of potential interrelated decay phenomena in building components.

The main feature of the model proposed is the automatic generation of pathological scenarios related to incompatibility phenomena between materials of building components. The model can be applied to any system we can describe by a topological and functional point of view; in the sense that model needn't refined geometrical description in order to produce its inferences.

Keywords: qualitative simulation, failure prediction, chemical-physical compatibility.

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1 Introduction

The issue we discuss in this paper is closely related to the problem of automated modeling for reliability analysis. The model we discuss is based on the concepts of graphical modeling because the structure of the graphical model is often quite easily derived from the structure of the system being modeled, or by using known engineering techniques, such as Failure Modes and Effects Analysis. Data about the components is often available in the form of engineering tests. The challenge lies in combining this information to the best advantage, not in creating new ways to collect and analyze new data. There have been a large number of techniques developed in the engineering community to produce system level reliability estimates (Henley and Kumamoto 1981). In particular, fault trees (which are very close to graphical models) have often been used for this problem. Through their statistical heritage, graphical belief models provide the same sorts of rigorous probability estimates that can be obtained from fault tree and other similar system level modeling techniques.

There are two roughly similar approaches to the automated support system in reliability modeling: Knowledge based model construction and Model fragment libraries. Knowledge based model construction (Breese at al.1991; Goldman and Breese 1992; Goldman and Charniak 1993) uses a conventional rule-based expert system to build the graphical model. Model fragment libraries (Almond et al. 1994) store fragments of models in an library or database. The modeler can then search through this library for factors or other model fragments which might be appropriate to the current model and re-use them in the new context. (Egar et al. 1992) propose using graph grammars for manipulating influence diagrams. (Habel 1992) discuss the use of directed hypergraph representation of a graphical belief network model, in order to manipulate graphs by a Hyperedge replacement grammars.

In this paper we propose an approach to the Bayesian networks assembling that involves Qualitative-Physics reasoning in the construction of the building component model. The key concepts and the terminology we use about Qualitative-Physics reasoning stem largely from the paradigm of 'Qualitative Process Theory' developed by (Forbus 1984) and used in (Naticchia 1991) as formalization environment of automated building details developing.

The approach drawn up allows to assign to assembling rules, similar to those used in knowledge based construction, a semantic related to the commonsense physical reasoning about the dynamic of processes interrelations. This feature permits to achieve the peculiarities of both approaches previously outlined. That is the propensity to a large level of automation in model construction and the capability of assembling complex fragments of knowledge about the behavior of system components. In order to represent this knowledge we employ a belief-network representation (Pearl 1988, 1996; Shachter 1990; Jensen 1996; Spirtes et al. 1993), that allows an effective formalization of model-based probabilistic reasoning about failures. For the same reason, the method here outlined is aimed to assemble a belief-network of the whole system we want to investigate. Once the model is available, a traditional MonteCarlo technique can assess the probability of any pathological decay phenomena of components, given a set of potential stresses.

In the following sections of the paper we firstly in short discuss the representation of and the features of the belief networks in their particular form of bayesian networks. Then we derive the outcomes of the assumption of knowledge distribution, concerning the specific fields of study involved in a reliability analysis. Finally we focus on the issue of qualitative reasoning as a mean to assemble those knowledge fragments.

2 Representing decay processes using Bayesian networks

Probability theory offers a well-understood language for describing uncertainty and for drawing inferences from partial knowledge. Thus, its methods are applicable to any task requiring inferences from incomplete information. Typical applications include diagnosis, forecasting, and classification. Many researchers have suggested using graphs to represent the structure of complex multivariate models. These models have gone by many names: Bayesian nets, influence diagrams, and graphical belief functions are fairly common. Each of these models differs slightly in terms of what kind of distributions they represent: Bayesian nets only allow probabilities, influence diagrams allow probabilities and utilities, graphical belief functions allow belief functions (and as a special case, probabilities). However, all of these types of models can be placed under a common framework, graphical belief networks. (Shenoy and Shafer 1990) describe the common framework in detail.). Belief networks are directed acyclic graphs in which the nodes represent variables of interest (e.g., the temperature of a device, the gender of a patient, a feature of an object, the occurrence of an event) and the links represent informational or causal dependencies among the variables. The strength of a dependency is represented by conditional probabilities that are attached to each cluster of parents-child nodes in the network.

Bayesian networks are the most popular of the current network based approaches. They are parameterized by conditional probabilities and offer the ability to explicitly represent dependencies among random variables at arbitrary levels of detail. Further, there are many published algorithms that reason with and about them. The networks possess straightforward interpretations, and this fact reduces the effort required to elicit the parameters as well as validate the encoded domain model.

Under a causal interpretation, the edges represent direct causal influences that are quantified by probability distributions associated with the nodes. While the interpretation of Bayesian networks was suggested by causal considerations, the formal semantics make no reference to causality. For example, in Fig. 1, x_1 and x_2 directly influence x_3.

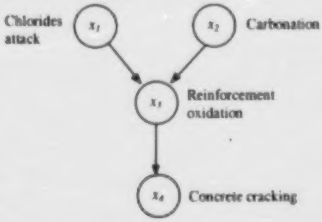

Figure 1: A simple bayesian network showing a fragment of cracking process in reinforced concrete

The influence of x_1 and x_2 on x_3 is quantified by the conditional probability distribution $P(x_3| x_1, x_2)$ associated with x_3. Similarly, the influence of x_3 on x_4 is quantified by the conditional probability distribution $P(x_4| x_3)$ associated with x_4. The variables x_1 and x_2 are, respectively, quantified by the marginal distributions $P(x_1)$ and $P(x_2)$. Finally, the principle of assuming independence, where possible, is a type of closed-world assumption used to delineate the semantics of the network.

The edges of the Bayesian network in figure 1 denote direct dependencies. Since , x_1 influences x_3 and x_3 influences x_4, it follows that x_1 indirectly influences x_4. However, there is no edge joining x_1 and x_4; therefore, x_1 has no direct influence on x_4. Thus, the influence of x_1 on x_4 is completely mediated by x_3. This means, in terms of probability theory, that x_1 and x_4 are conditionally independent given x_3. This conditional independence is denoted by $I(x_1, x_4|x_3)$. Similarly, since no variable influences both x_1 and x_2, they are marginally independent, written $I(x_1, x_2|\varnothing)$. The derivation of such I statements is a manifestation of the default rule that assumes independence whenever it is consistent with the network.

Strictly speaking, it is not accurate to say that arcs represent dependence. More accurately, in a belief network, it is the absence of arcs that represents conditional independence. Any pair of attributes (nodes) which are not directly connected are assumed to be conditionally independent under the Markov conditions (since it reflects the Markovian condition for the state transitions that is each state is rendered independent of the past, given its immediately preceding state). However, that distinction will not be important to our discussion here.

Because Bayesian networks allow us to capture information about conditional independence, they make it possible to compactly represent probability distributions. The probability distribution corresponding to a Bayesian network may be specified by giving conditional probability distributions for each node in the network. The conditional probability distribution gives the probability of the random variable taking on each possible value conditioned on each combination of values of its parent nodes. If the node has no parents (a 'root' node), one specifies a prior distribution over the values of that node. Thus, the joint distribution over all variables is assumed to be the product of the distributions associated with the nodes.

$$P(x_1,x_2,x_3,x_4)=P(x_1)P(x_2)P(x_3 \mid x_1,x_2)P(x_4 \mid x_3)$$

from which all probabilistic queries, involved in diagnostic reasoning, can be answered coherently using probability calculus. In fact because the Bayesian network determines the joint probability distribution, we can, in principle, use a Bayesian network to compute any probability of interest.

However while in diagnosis the issue is that of finding the most likely explanation for the evidence, in prediction reasoning, performed during design phase of new systems, we might want know what are the most likely causes of failure or decay. Differently to the diagnostic process, in this case the pathological state doesn't represent an evidence because it hasn't yet occurred. Moreover in the early phase of design it's difficult even to suggest possible failure scenarios because of the vagueness of the functional and causal structure of the system. Nevertheless, once a Bayesian network about the system behavior is available, it could be used effectively in order to envision the reliability performance of the system, assessing, by means of a stochastic simulation, the probability of the fast decay of a component, given a set of potential stresses. For example with the network in figure 1, one can assess whether or not the cracking probability of a reinforced concrete element is too high, given the probability that the element will be in contact with an aggressive environment and the probability that a carbonation process will be in highly evolved.

Thus the main problem in reliability analysis, regards the construction of the Bayesian network prior to availability of experience data regarding the behavior of a system.

3 Assembling Bayesian networks by means of Qualitative Physics reasoning

In the early phase of design any attempt of envisioning the Bayesian network of the whole system is impractically large (aside from any other issues, just consider the number of chemical and physical phenomena potentially involved in reliability system performance and the consequent expertise needed).

As previously stated, a plausible way to approach the intractability of the problem is the strategy of assembling ready-made 'chunks' of knowledge in order to construct a complete model of system behavior dealing directly with few variables and relationships. With this approach, fragmented structures of causal organizations could be assembled as needed, from a stock of existing functional building blocks, ideally like a jigsaw puzzle, as showed in Figure 2.

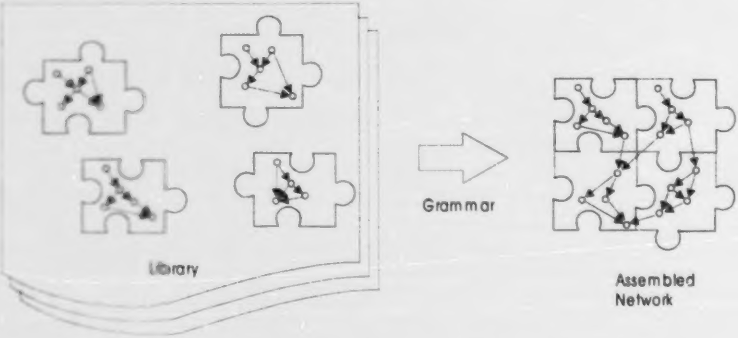

Figure 2: Construction of Belief Network for a new problem by assembling model fragments stored in a library.

For example consider the network of figure 3, related to the cracking phenomena in reinforced concrete elements. It may be assembled from several Bayesian networks, one specializing in the experience related to chemical combinations, another in the properties of concrete, another in the properties of steel, and so forth. Such specialized networks can be stored permanently in a library, from which they are drawn and assembled into the structure shown in figure 3 only when a specific problem presents itself, for example, to determine whether the carbonation or high porosity of concrete could explain why reinforcements are affected by an intense corrosion. It means that we only need to define probability function mechanisms by which graph fragments that share common variables are connected maintaining analytical coherence of assembled model.

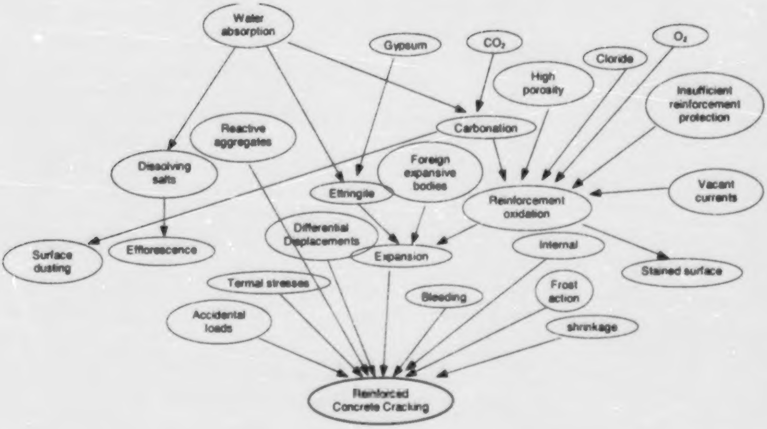
Water absorption
Gypsum
CO_2
O_2
Cloride
High porosity
Insufficient reinforcement protection
Reactive aggregates
Carbonation
Dissolving salts
Foreign expansive bodies
Ettringite
Reinforcement oxidation
Vacant currents
Differential Displacements
Internal
Surface dusting
Efflorescence
Expansion
Stained surface
Termal stresses
Frost action
Bleeding
Accidental loads
shrinkage
Reinforced Concrete Cracking

Figure 3: A fragment of the Bayesian Network representing influences among variables involved in cracking problem of reinforced concrete. For the sake of simplicity conditional probability values are not showed.

Regarding to this problem we are working on the definition of a grammar, namely a collection of rules, capable of supporting the process of constructing a graphical model and providing method for integrating high-level and detailed views of a graphical model. In remainder part of the paper a way to use the paradigms of qualitative reasoning in physical domains is explored in order to define the construction grammar.

The grammar is tailored to be applied iteratively to the domain of interest, anytime applicability conditions are satisfied, in order to produce a belief network consistent with the domain description. Strictly speaking a grammar consists of three main components: a vocabulary of non terminal and terminal symbols, a set of writing rules and an initial pattern. The vocabulary is partitioned into terminal and non terminal symbols in order to distinguish completed structures, which belong to the language of grammar from intermediate structures. In the case of network grammar, we consider terminal symbols the constitutive elements of bayesian networks, that is the domain random variables depicted as nodes and the probabilistic influences depicted as arcs and consider non terminal symbols, the data about the elements the studied system and their reciprocal relationships (system topography). A writing rule consists of a precedent, which specifies the conditions under which the rule applies to a target structure and a consequent that specifies the effect of applying the rule to the target. The grammar can be applied in a rule-based production system, which properties are described in large body of literature. Such a tool can drive interactively the process of assembling the network, suggesting to pinpoint the application of a rule to a specific part of the partially assembled net.

The initial pattern allows the generative process to start. In the model proposed the role of initial pattern is played by the description of the system topography, that consists in the description of component properties and their reciprocal interrelations.

Each component in a system is related to the other components in a specific manner, and identical components may have different characteristics in different systems. The interrelations and the topography are found by examining mechanical couplings, the physical location of components, the functional relationships among components. These can be best expressed by a system schematic. For example, interactions between two closely located materials are possible in case of chemical incompatibility.

At present we consider two kind of description level of the system configuration: the positional structure and functional structure.

Perhaps the most important description level of the system topography is the positional one. In this level the component connectivity is yielded representing the structure of positional relationships between components. As described below, this structure will be then translated, by means of grammar rules, assigning to each relationship a physical semantics, for example pertaining the transfer capability of some decay agent. To illustrate the nature of positional structure consider the detail shown in figure 4 where component connectivity is expressed by a graph whose nodes represent the components and whose arcs represent positional relationships.

Another important description level of the system topography is the functional one. In this level the component interrelations pertain the way of functional accomplishment. For example, in the detail of figure 4, the framework of structural interrelation pertains especially the structural relationship between elements.

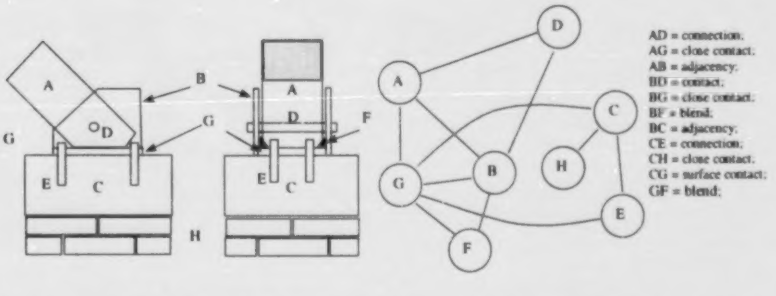

Figure 4: Detail of the seat of a timber joist and the related positional structure

The quality of system topography identifying, depends on the competence of user in perceiving the relevance of reciprocal influences of parts as for phenomena of interest.

In figure 4 each part of the configuration is represented by a unit of information concerning causal structure of typical failure of that part. In this manner, any component can be modeled by a bayesian network, which describes informational or causal dependencies among the variables of interest for the specific failures of the component. The modeler using the library of fragments needs to know what given fragment is used for and how to connect it to other variables in the model. For example a model fragment describing the reliability of the timber joist, might be specific to just that component. Nevertheless it shares some common issue with any timber component since made of the same material. Thus a set of 'replacement rules' offer the possibility of linking overviews and detailed views of a graphical model.

For example if the component of figure 5a C_1 is independent of C_2, we can factor the probability distribution over the variables, C_1; C_2 and E, into three pieces as follows:

$$p(C_1,C_2,E)=p(E\mid C_1,C_2)\cdot p(C_1)\cdot p(C_2)$$

(Habel 1992) defines the concept of a hyperedge replacement grammar; that is nothing more than a collection of grammar rules describing when you can replace a hyperedge with a more elaborate graph fragment. Figure 5b shows the interpretation of the directed graph in a correspondent hypergraph representation of this factorization where the square hyperedges represent the factors of the joint probability distribution. Figure 5c shows as a replacement rule substitutes the hyperedge with a fragment

Sometimes the appropriate input variables will already be in place in the graph, as C_2 in Figure 5c. To handle these cases, we need an another graph manipulation which merges repeated variables, as in Figure 5d.

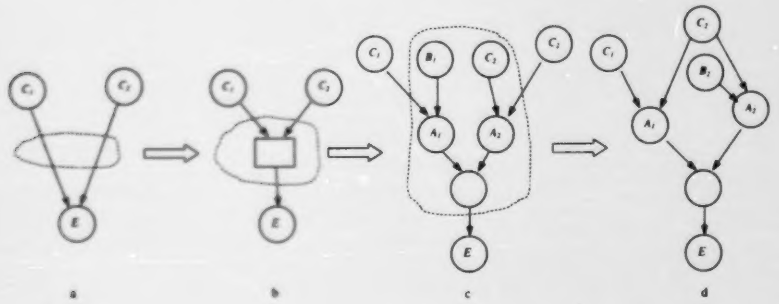

Figure 5: Development of a network fragment by means of replacement rules

After the replacement nodes in topography description, with corresponding fragment graphs and the further replacement, possibly of parts of these with more detailed one, the database of system contains disconnected 'chunks' as depicted in figure 6. Reasoning about the physical world requires reasoning about the kinds of changes that occur and their effects when things are connected constructing a system. Then the goal of 'process rules' is to represent the knowledge experts have about the physical processes. Due to the semantic given to these rules, they take the name of 'processes' and are closely related to 'qualitative process theory' formalization (Forbus 1984). The structure of a process is showed below:

Process (Galvanic corrosion)

Parts: object a, object b

Preconditions: contact (object a, object b), a (material, metal), b (material, metal), a(material) ≠ b(material).

Quantity Conditions: abs(a(Potential) – b(Potential)) > 0,1 V;

Relations: graph a (different metal contact) ≡ graph b (different metal contact)

Influences: graph a,b (different metal contact) = 1.0

The slots Parts, Preconditions and Quantity Conditions constitute the precedent of the rule, the slots Relations and Influences constitute the consequences.

In the case of 'Galvanic corrosion', the rule engine (the manager of rule firing) searches in the database two objects (network fragments), assigning them to rule variables *a* and *b*, which are linked by a 'contact' relationship in the topography description and are constituted of different metals. Then, if the difference of their chemical potential is greater than 0,1 Volts the rule 'Galvanic process' is fired. The rule, as stated in the slot 'Relations', produces its effects unifying the nodes/variables 'different metal contact' belonging to *a* and *b* object networks; moreover it updates the unified node/variable to 'true' putting in it the value 1,0. In the example of figure 4, objects 'G' and 'E' are matched with variables a and b of the process rule. An analogous procedure can be described for the process 'Corrosion by differential aeration' which however simply updates the node/variable 'surface contact' of element 'G' in figure 4 to 'true' putting in it the value 1,0. The results of binding are showed in figure 6 as dashed arcs between nodes of network fragments.

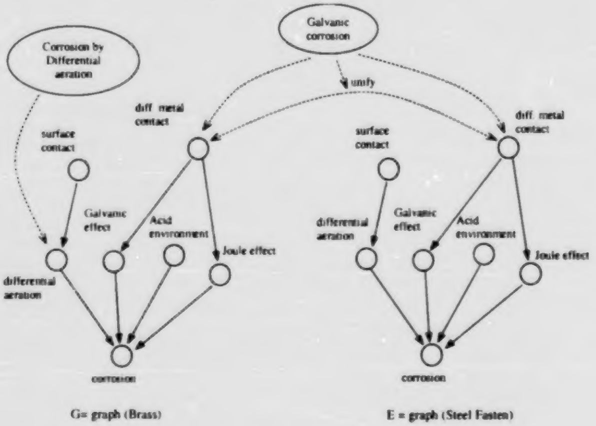
Galvanic corrosion
Corrosion by Differential aeration
unify
diff metal contact
surface contact
diff metal contact
surface contact
Galvanic effect
Acid environment
Joule effect
differential aeration
Galvanic effect
Acid environment
Joule effect
differential aeration
corrosion
corrosion
G= graph (Brass)
E = graph (Steel Fasten)

Figure 6: The binding of disconnected fragments by means of process rules

4 Conclusions

We have discussed a framework to support assessment of chemical-physical compatibility problems in the early phase of design. The model presented here is under implementation in order to carry out an automated assistant tool. The main feature of the model proposed is the automatic generation of pathological scenarios related to incompatibility phenomena between materials of building components.

The model proposed enables a kind of distributed engineering of reliability analysis. For example, one engineer (a systems level reliability expert) would build small model fragments describing the interaction among a few components. Another engineer (a testing and purchasing expert) would develop model for the various commonly used components from the usual vendors. A third engineer (a system designer) would then assemble the model from the fragments created by the other two engineers, drawing on their expertise in the form of stored model fragments.

5 References

Almond, R., Bradshaw, G.J., and Madigan D., (1994) Reuse and Sharing of Graphical Belief Network Components. in Cheeseman P. and Oldford W. (eds.) *Selecting Models from Data: Artificial Intelligence and Statistics IV*, Springer-Verlag. pp. 113-122.

Breese, J. S., Goldman R., and Wellman M. P., (1991) Knowledge-Based Construction of Probabilistic and Decision Models: An Overview. *Workshop presented at AAAI-91.*

Egar, J.W., Puerta, A.R., and Musen, M.A., (1992) Graph-grammar assistance for modelling of decision. In *Proceedings of the Seventh Banff Knowledge Acquisition for Knowledge-Based Systems Workshop*. pp. 1--19. Banff, Alberta, Canada.

Forbus, K.D., (1984) Qualitative Process Theory. In *Artificial Intelligence Vol. 24*. pp. 85-168.

Goldman, R. and Breese J.S., (1992) Integrating Model Construction and Evaluation in Dubois et al. (eds). *Uncertainty in Artificial Intelligence, Proceedings of the Eighth Conference*, Morgan Kaufman. pp. 104-111.

Goldman, R., and Charniak E., (1993) A Language for Construction of Belief Networks. *IEEE Pattern Analysis and Machine Intelligence Vol. 15*. pp. 196-208.

Habel, A., (1992) *Hyperedge Replacement: Grammars and Languages*. Springer-Verlag.

Henley E. J., and Kumamoto H. (1981) *Reliability engineering and risk assessment*, Prentice Hall Inc., Englewood Cliffs, N.J.

Naticchia, B., Lemma, M., and Fornarelli, A., (1991) A computer assisted system to design construction elements. In *Proceedings of International Association for Housing Sciences Congress IAHS 1991*, Ales, France.

Pearl, J. (1988) *Probabilistic Reasoning in Intelligence Systems*. Morgan Kaufmann, San Mateo, CA.

Pearl, J. (1996) Causation, action, and counterfactuals. In Y. Shoham, editor, *Theoretical Aspects of Rationality and Knowledge, Proceedings of the Sixth Conference*, pp. 51-73. Morgan Kaufmann, San Francisco, CA.

Shachter, R. D. (1990) Special issue on influence diagrams. Networks: *An International Journal*, Vol. 20 (5).

Shenoy, P.P. and Shafer, G. (1990) Axioms for Probability and Belief-Function Propagation. In *Uncertainty in Artificial Intelligence, Vol. 4*. pp. 169-198.

Spirtes, P., Glymour, C., and Schienes, R., (1993) *Causation, Prediction, and Search*. Springer-Verlag, New York.

137 SERVICE LIFE PREDICTION FOR FLOOR COVERINGS

J. Paulsen

Building Materials, Royal Institute of Technology, Stockolm, Sweden

Abstract

A Life Cycle Assessment on floor coverings in the public and service sector is under progress in Sweden, with emphasis on the usage phase. The result is sensitive to the predicted service life and it is highly desirable with a good estimate of the actual service life. But, service life prediction for a specific type of floor covering has shown difficult. However, an important topic is to detect the relation between the type of service life and the type of premise where the floor covering is installed. Five different types of service life are suggested, and further, it is suggested that technically and aesthetically requirement play an important role, strongly influenced by the economy. The public and service sector is also divided into five different types of premises and related to the different types of service life. It is showed that the prediction of service life for floor coverings can not be done with data on floor coverings only. Further information is required concerning maintenance methods and building related data. No intentions have been made to estimate the length of the different service life.

Keywords: Floor coverings, Service life, Public sector, Life cycle assessment, Usage phase, Swedish building stock

1 Introduction

To predict the service life for a specific type of floor covering has shown to be difficult. When a Life Cycle Assessment (LCA) has to be carried out on floor coverings with emphasis on the usage phase, one important factor is the length of the service life (Paulsen, 1998). Earlier case studies on floor coverings have shown that there are difficulties in estimating this factor (Jönsson 1994, Pottring

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1993, Lundblad 1994, Günther&Langowski 1997). Rough estimates have been used. Common for those case studies is that the emphasis has been on the production stage and the waste treatment stage. The usage phase has been excluded or only briefly handled. Further, the emphasis has been on the floor covering as a material regardless of where it is placed.

A case study on floor coverings in the public and service sector in Sweden is under progress (Paulsen 1998). It is an attempt to carry out an LCA with emphasis on the usage phase. The result is sensitive to the predicted service life and it is highly desirable with a more sophisticated estimate.

Common for the floor coverings in the public and service sector is that professional cleaning companies maintain them to a large extent.

The aim of this paper is to define different types of service life for floor coverings in the Swedish building stock. Further, focus is on the premises in the public and service sector. Here the economy plays an important role and an attempt is done to describe the interrelation between the different types of service life with focus on the economy. No intention has been made to estimate the length of the different types of service life.

2 Types of service life for floor coverings in Sweden

To determine the service life for a floor covering, it is necessary to define the different types of service life times. In this paper, five different types of service life are suggested for the public and service sector (Lidström 1998, Paulsen 1998). However, the same types of service life can be used for other parts of the building stock:

1. Technical (functional) service life
2. Aesthetic service life, (type 1)
3. Aesthetic service life, (type 2)
4. Economic service life
5. Ecological service life

For the same application, these five service life can differ widely. One major problem is to decide which one determines the actually service life for the floor covering. In the following, a short description of the different service life is given.

2.1 Technical service life

The technical service life can be seen as the maximum service life for a floor covering. In the definition of this service life it is understood that it can not be exceeded without serious consequences. The technical service life depends on the functional demands on the floor covering. Normally, different kinds of properties and demands for a floor covering are set in relation to the actual type of premises. Demands can for example be set concerning cleanability and slipperiness etc.

2.2 Aesthetic service life, type 1

The aesthetic service life is to a large extent dependent on the form of activity in the premises in combination with trends and fashion at the current time. The choice of aesthetic service life is based on a subjective valuation and thereby depending on individual opinions. No rules or guidelines exist to estimate the aesthetic service life. One theory is that the change of floor coverings in certain types of premises is following the change of tenant. This normally occurs when a new tenant wants to set his/her own personal expression in the premises.

2.3 Aesthetic service life, type 2 (Upgrading service life)

The upgrading service life is not a common concept but a preliminary definition to cover a situation where the floor covering is replaced because of a larger upgrading of the premises, involving more than one actor. For example, if a department store is upgraded, the floor covering is also upgraded or replaced to make an attractive environment together with the rest of the premises. It can be discussed if it shall be regarded as an aesthetic service life, but in this case the service life is decided independent of the condition of the floor covering (Lidström 1998).

2.4 Economical service life

The economical service life is a monetary relation between the costs for maintenance at the current time and the cost for replacement of a new floor covering with reduced maintenance cost. The economical service life only exist under the assumption that maintenance cost increases in time because the floor demands an increasing maintenance with age, or because new and cheaper maintenance methods are developed which are not applicable on the current type of floor covering. There also exists a relation between the economic and aesthetic service life, which will be described later.

2.5 Ecological service life

The ecological service life is determined in the similar way as the economical service life, but with emphasis on the environmental factors such as energy use, global warming and acidification. One major problem is that the environmental impacts are a multi dimensional parameter, which is very difficult to normalize. Even the individual environmental impacts are normally hard to estimate in the present-day situation due to lack of data.

3 Dividing the premises into different types (regarding the Swedish building stock)

The Swedish building stock comprises approximately 647 million m^2 of floor covering. See table 1 (Bejrum 1997).

Table 1: Swedish building stock 1995

No	Type of building	Surface (Million m²)	Share (%)
1	Houses	226	35
2	Multiple-flat buildings	154	24
3	Industry	110	17
4	Public buildings, administration	85	13
5	Offices, stores, hotels	72	11
	Total	**647**	**100**

The largest group of buildings is dwelling houses (type 1 and 2 in table 1). Here, it is very hard to categorize the aspects or valuations, which is fundamental for the determination of the service life. The actors in this group are non-homogenous, with a broad range of individual opinions about requirements on performances concerning floor coverings.

For the industrial sector (type 3 in table 1), it can be expected that the requirements are high on the technical performance while the aesthetic aspects have a lower priority.

In the public and service sector (type 4 and 5 in table 1), large expenses are connected with the maintenance compared to buying and installation of new floor coverings. Here it can be expected that the economical service life have a dominant influence. Table 2 shows Swedish trade figures (1995) for floor coverings to illustrate the investment costs (GBR 1996).

Table 2: Investment costs (incl. installation) for 23 million m² floor coverings

	Trade in Sweden 1995 (million SEK)	Average costs per m² (SEK)
Resilient floor coverings	3,000	130
Installation	700	30
Total	3,700	160

In Sweden, the maintained surface for the public and service sector was 150 million m² floor covering in 1996. The costs for cleaning were 17,000 million SEK. (SG 1997). A good estimate is that around 70% of the cleaning costs are related to the floor covering. This gives an average cost of approximately 80 SEK per m² a year. In relation to the investment cost for a new floor covering, the maintenance costs are considerable. In other words it can be said, *"It is quite cheap to buy a floor covering, but expensive to own it"* (Forslöw 1989).

The public and service sectors (type 4 and 5 in table 1) represent approximately 24% of the Swedish building stock. Unfortunately, no inventory data exists concerning the areas for a further sub grouping into premise types (Laurila 1998). However it is desirable to make a new grouping for these premises and their relation to the type of decisive service life to support the LCA study mentioned earlier (Paulsen 1998). A suggestion is given in table 3 (Lidström 1998; Franzén 1998).

Table 3: Type of service life related to type of premise in the public and service sector

Type of premises	Type of service life	Area in million m^2
Schools	Technical	No data
Department stores	Aesthetic type 2	No data
Hospitals	Technical	No data
Offices (private)	Aesthetic type 1	No data
Offices (public)	Technical	No data
Total (approximately)	----	**150**

4 Interrelation between the different types of service life times

It is doubtful if the different types of service life times can be considered as independent of each other. For premises in the public and service sector, it has earlier been suggested that the economy is a very dominant factor. For these types of premises it would be interesting to discuss the interrelation between the different kinds of service life.

In figure 1, a suggestion is given to describe how the economy influences the different types of service life in the public and service sector.

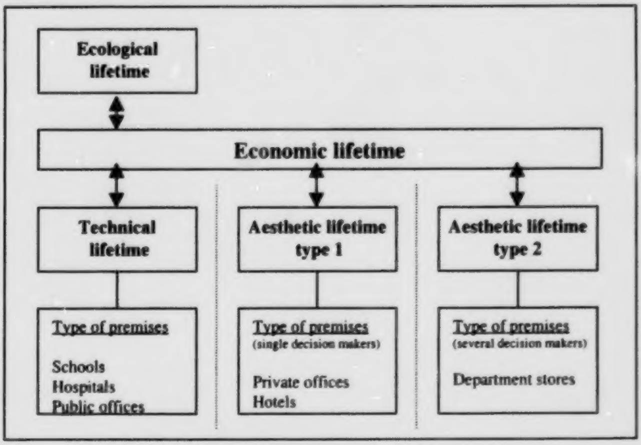
Ecological lifetime
Economic lifetime
Technical lifetime
Aesthetic lifetime type 1
Aesthetic lifetime type 2
Type of premises
Schools
Hospitals
Public offices
Type of premises
(single decision makers)
Private offices
Hotels
Type of premises
(several decision makers)
Department stores

Fig. 1: Economic influence on service life

As illustrated in figure 1, the economic service life can be expected to play an important role.

For schools, hospitals and public offices, the technical service life can be considered as the determining factor. However, the economy plays a considerable role because one of the most important technical demands on this kind of floor covering is that it is easy to clean. Indirectly, this is an economic demand, because the costs for cleaning are strongly dependent on the time required for cleaning the surface. Parallel to economic aspects, the ecologically aspects can be considered. Unfortunately the foundation for this is not yet properly developed concerning the usage phase (Paulsen 1998).

For private offices and hotels, the aesthetic service life (type 1) normally can be considered as the determining factor. Nevertheless, this can also be considered as an indirect economical service life. In this kind of premise, the image is an important function for the floor covering. The aesthetic service life (type 1) can be set in relation to the market value of the premise. If the floor covering does not satisfy the customer's demands on aesthetics, the premises no longer preserve the possible market value, and thereby the aesthetic service life for the floor covering can indirectly be seen as an economical service life. The aesthetic service life could be assumed to be strongly connected to the change of tenant.

Service life in department stores can be more complicated to determine. In this paper the service life is suggested to be dependent on aesthetic aspects. Similar to the private offices and hotels, the aesthetics is an important function for the floor covering. Also there is a strong interrelation with economic aspects. The premises have to be attractive for the tenants. However, here the situation differs from hotels and private offices in an important issue: in a department store, several actors have to accept an aesthetic level, which is acceptable for the majority.

As suggested here, it seems possible that the service life is dependent on a common decision by several actors, with primary focus on the premises as a whole and not only the state of the floor covering.

For the premises with the two described types of aesthetic service life, it seems difficult to express ecological aspects parallel to the economic aspects. In that case, the value should lay in the value of an environmental profile for the tenant in the premises.

5 Factors influencing the service life in the public and service sector

As earlier mentioned, different types of service life can be related to different types of premises in the public and service sector. Important topics are technical and aesthetic demands, influenced directly or indirectly by the economy. However, both technical and aesthetic performances are further influenced by other factors (GBR recommendations 1996). Three main factors are properties of the floor covering, maintenance methods and building related factors. Those will be described further in the following.

5.1 Floor covering related factors

5.1.1 Technical product properties

Different kinds of floor covering have different kinds of properties concerning the wear-layer thickness, resistance to wearing, etc. which all give information about circumstances affecting the life time. The main part of floor covering types can be divided into different types of wear classes, with information about appropriate type of premises.

By installation of a material (floor covering) with a higher wear-class than the premise demands, the service life can be extended (and vice versa).

5.1.2 Upgrading

Some kinds of floor covering can be upgraded, when the wear has destroyed the technical or aesthetic function. Wood and linoleum floorings can be grinded and surface treated. Polyurethane (PUR) layers on plastic coverings can be renovated and pieces of coverings can be replaced. In that way, a floor covering can achieve a considerable extension of the service life.

5.2 Maintenance

Correct care and maintenance, are prerequisites for a long service life for the floor covering. Incorrect maintenance can damage the floor covering in only a few months. Further, the periodical maintenance can protect the floor covering against wearing, and reduce the frequent cleaning costs.

5.3 Building related factors

5.3.1 Location in the building

On ground level the wear of floor covering is larger than higher up in the building, which demands a hardwearing floor covering and more intensive care. The wear on floor coverings also varies considerably over the floor surface. In door openings and walk-traffic intensive areas, the wear is much larger than e.g. in corners.

5.3.2 Design of the entrance

Correct performance of wipe-off zone in the entrance is a very important factor concerning the service life for floor coverings.

5.3.3 Traffic intensity and type of load

In an office building the service life of a floor covering is shorter in a corridor than inside an office, if the same type of covering is used in both places.

5.3.4 Geographical location of the building

The geographical location of the building also plays an important role. The outdoor environment will to a large extend influence the amount and type of dirt in the building. However, here it is important to analyze this topic in combination with the design of the entrance.

6 Discussion and conclusions

As described above, it is almost impossible to predict the service life of a floor covering only considering the floor materials. Nevertheless, when the application is known the range of information increase and a better estimate can be done.

This paper suggests several relations between the required type of service life for floor coverings and the type of premise. For premises in the service and public sector, the economic aspects can be expected to have a strong influence when attempts are done to predict the service life. This is due to the fact that the maintenance costs are very high compared to the investment costs for new floor coverings. The maintenance costs are approximately higher for two years of maintenance than the investment costs for a new floor covering.

Technical requirements often reflect the demands on low maintenance costs, and aesthetic demands often reflect the premises economical market value. Ecological service life for floor coverings can theoretically be calculated and regarded similar to the economical aspects for floor covering, but the foundation has not yet been properly developed.

7 References

Bejrum, H (1997) Real estate economics, KTH, Sweden, personal communication

Franzén, T. (1998) Partena AB, (Swedish cleaning company) personal communication.

Forslöw, G (1989) Maintenance costs for floor coverings. *Golvets driftsekonomi* In Swedish, Tarkett

GBR. (1996), Swedish branch organization for floor coverings, *Golvbranschens riksorganisation, Golvtekniska pärmen.* In Swedish, technical report.

Gunther, A and Langowski, H (1997) LCA study on resilient floor coverings. *The International journal of Life Cycle Assessment*, vol. 2, No. 2. pp 73-80

Jönnson, Å. (1995) Life cycle assessment on flooring materials, Licentiate thesis, CTH, Sweden

Laurila, J: (1998) SCB, Statistic Sweden (Statistiska Centralbyrån) personal communication.

Lidström, R. (1998):ISS, Sweden AB, (Swedish cleaning company) personal communication.

Lundblad, D: (1994) Environmental impacts from maintenance of floor coverings (Miljöpåverkan av golvvård) In Swedish, KTH, Trita-Byma 1994:7

Paulsen, J. (1998) Life Cycle Impact of Floor Coverings-A Model for the Contribution of the Usage Phase, proceedings of the CIB World Building Congress Gävle 1998

Pottring, J. (1993) The environmental life cycle analysis of some floor coverings, 1993, Journal of cleaner production, vol. 3, No 4.

SG (1997) Swedish council of floor materials and floor maintenance (Svenska golvrådet, DUS A flik 8) In Swedish, Trade statistics.

138 EVALUATION OF FUNCTIONALITY OF BUILDING COMPONENTS APPLICABLE TO SERVICE LIFE DESIGN

M.G. Rejna

Polytechnic of Milan, Italy

Abstract

This paper proposes a method to evaluate the functional compliance requirements of building components as a tool to estimate the propensity of a given design to attain functional requirements over their service-life. This method is part of a broader research project dealing with developing methods for durability evaluation of materials and building components. The method is derived from the analysis of the functional aspects of building components and uses four essential aspects of compliance to assess an overall or global compliance index: functional, fabrication, inherent, and compatibility. Each of these aspects is described and detailed methods for calculating the respective criteria are given. An example of the proposed functional compliance analysis method has been applied to a set of two non load-bearing external wall components. The proposed methodology offers an additional tool for new building component design in that it potentially can facilitate the selection and optimisation of building components for durability.

Keywords: functional compliance analysis, functional compliance, fabrication compliance, inherent compliance, compatibility compliance.

1 Introduction

The development of methods to determine the functional compliance of building components in design is based on previous research undertaken at the Building Department (DISET) of the Polytechnic of Milan (Maggi 1992; Maggi 1993; Rejna 1995). This design tool evaluates one of the two parameters of durability, namely a components' propensity to remain in a functional state

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throughout its life given that it has been fabricated and assembled according to recommend and approved standards and recognised construction practice.

An estimate of the propensity of a component to reach its mean time before failure (MTBF) can be made by analysing how the component is designed in relation to its related elements and materials, the complexity of the components, the degree of attention paid to fabrication and assembly, to the inherent physical characteristics of the elements, and to compatibility between adjacent materials. For the purposes of this paper, the process by which the analysis is completed is referred to as functional compliance analysis. An evaluation using functional compliance analysis provides a qualitative means of determining, in an objective a manner, the degree of propensity of different components (assemblies) to deteriorate over their service life. It must be emphasised that the values obtained using this method, even if applied on a specified set of building components, will necessarily change if the environmental conditions change. That is to say, if the agents of deterioration affecting the functional parts of the building assembly change the results of this analysis will also be different.

The proposed method takes into consideration four key aspects: The first aspect regards the *functional compliance* of the component and is used as a method to verify the degree to which there exists the distribution of analytical functions. This analysis is undertaken on the functional model used in the design of the component.

The second aspect regards analysing, at the design stage, the extent to which the built component will meet design intentions, and provides a measure of the degree to which functional requirements in the fabrication and assembly of the component can be met. It is referred to here as *fabrication* (and assembly) *compliance*.

The third aspect regards analysing designed components as a means to highlight dimensional changes that are likely to occur during the life of the component due to interaction with the environment (i.e. micro-climate) in proximity to the component. Evidently, these changes would be different for each functional element of the component and might highlight risks of early damage of individual elements during its service life. This aspect is referred as the *inherent compliance* of the component.

The final and fourth aspect regards analysing designed components as a means to verify possible chemical and/or physical incompatibilities among different materials that collectively define the component. The presence of these kind of incompatibilities is potentially harmful to the overall durability of the building component and the number of such incompatibilities offers a measures of the value of the *compatibility compliance* of the component.

2 Proposed method to evaluate the degree of functional compliance

This method is based on an analysis of a component's working "model" in which the components have been discretised into elements and materials as required for the analysis. This analysis allows verifying specific aspects to which the proposed criteria are applied. Each non-dimensional value resulting from the application of the four key compliance aspects mentioned previously are qualitative indicators of the propensity of a component to attain a degree of functional compliance. Criteria appropriate to the compliance aspect are grouped in classes and are applied to the design of a building component. P_i is a measure of the degree to which a single criterion meets a compliance index, in which I_i is derived from P_i such that the index I_i, ranges between 1 and 100.

$$I_i = \left(1 - \frac{P_i}{100}\right) \cdot 100 \tag{1}$$

I_j, global indicator for the j-th class, is computed as the average of all the indicators I_i in the same class, where n is the number of criteria in a given class:

$$I_j = \frac{\sum_{1}^{n} I_i}{n} \tag{2}$$

The value of each aspect of functional compliance (A_{fu}, A_{fa}, A_{in}, A_{co},) is computed as the average of individual indicators, I_j, evaluated for all the criteria classes belonging to the respective aspects of compliance, over, k, the number of criteria classes for each compliance aspect:

$$A_{(fu,\, fa,\, in,\, co)} = \frac{\sum_{1}^{k} I_j}{k} \tag{3}$$

The evaluation of the degree of global compliance of the building component design is computed as the average of values of the four specific aspects of compliance:

$$A_g = \frac{A_{fu} + A_{fa} + A_{in} + A_{co}}{4} \tag{4}$$

This is necessarily of synthetic evaluation and is less significant than an analytical evaluation related specifically to each compliance aspect. Nonetheless, the criteria could also be seen as useful design guide to help determine potential instances where of there exists a propensity for premature failure of building components.

An example is provided that applies the method of functional compliance analysis to evaluate the propensity of a set of non-load bearing vertical perimeter walls to meet functional requirements in the terms of functional, fabrication, inherent, and compatibility compliance as well as the global compliance.

2.1 Functional compliance

The evaluation of the functional compliance is based on the functional model of the designed component, that is to say on a model of the component obtained using the functional analysis (Rejna, 1998). The global functional model of the component is evaluated according to the criteria provided in Table 1.

Table 1: Calculation of criteria for functional compliance

Criteria Class	Criteria	Calculation of criteria	I*
Model's simplicity (A)	Structural simplicity	Number of functional locations (P_{A1})	I_{A1}
	Functional simplicity	Number of qualitatively diversified analytical functions (P_{A2})	I_{A2}
	Location simplicity	Number of basic function performed by single location (P_{A3})	I_{A3}
Functional fatigue (B)	Mean fatigue	Number of distributed analytical functions (P_{B1})	I_{B1}
	Critical fatigue	Maximum amount of analytical functions located in the same location (P_{B2})	I_{B2}
	Variability fatigue	Dispersion of analytical functions load (P_{B3}) $P_{B3} = P_{B1} - \left(\frac{P_{B2} - P_{Bmin}}{2}\right)$ (5) P_{Bmin} = minimum amount of analytical functions located in one single spot of the same model	I_{B3}
	Balance fatigue	Balanced distribution of functional load (P_{B4})	I_{B4}
Functional distribution (C)	Serial distribution	Number of analytical functions related to a basic function, assigned to two or more different locations (P_{C1}) $P_{C1} = \sum_{i=1}^{r} n_{s_i}$ (6)	I_{C1}
	Parallel distribution	Number of analytical functions related to a basic function, repeated on different locations (P_{C2}) $P_{C2} = \sum_{i=1}^{r} n_{p_i}$ (7) r = number of basic functions relevant to the building component class	I_{C2}

* index

2.2 Fabrication (and assembly) compliance

The evaluation of the fabrication compliance is derived from an analysis of the complexity of design of the building component. It is based on a forecast of possible differences between the design of the component and that of the fabricated component due to fabrication and assembly errors during construction of the component. The dimensional complexity of a given component is related to the objective or "base-case" model; a model to which different components can be compared on an "objective" basis. This objective model is evaluated according to the criteria provided in Tables 2 and 3.

Table 2: Calculation of criteria for component complexity

Criteria Class	Criteria	Calculation of criteria	Index
	Materials diversity	Weighted number of different types of materials of the building component (N_M)	I_M
	Complexity of components	Number of functional elements of the building component (N_o)	I_O
Relational complexity	Relational complexity in X direction	Weighted number of interfaces of the building component in X (I^X)	
	Relational complexity in Z direction	Weighted number of interfaces of the building component in Z (I^Z)	
	Relational complexity in Y direction	Weighted number of interfaces of the building component in Y (I^Y)	
		$P_R = I^X + I^Z + I^Y$ **(8)**	I_R

Table 3: Basis for calculating fabrication compliance

Calculation Criteria		
Materials diversity index $N_M = \sum_{i=1}^{M} p_i$ **(9)**	$p_i = 0.5$ in factory production; $p_i = 1$ in site production; M = Number of different types of materials of a building component.	
Relational Complexity $N_{ij}^{Y.Z} = \frac{\left(F_{ij}^{Y.Z} \bullet M_{ij}^{Y.Z} \bullet E_{ij}^{Y.Z}\right) f_{ik}^{Y.Z}}{MS_j}$ **(10)** $N_{ij}^{Y} = \frac{\left(F_{ij}^{Y} \bullet M_{ij}^{Y} \bullet E_{ij}^{Y}\right) f_{ik}^{Y}}{MS_j}$ **(11)**	i = i-th type of interface of the j-th functional element, in X and Z directions of the building component within the surface iterative modulus MS_j of the functional element; in Y direction of the k-th interface between functional elements of the building component within the highest surface iterating modulus, Ms_{kmax}, between those of the two contiguous functional elements of building component; k = k-th interface, in Y direction, between functional elements of the building component; f_{ij} = frequency of i-th type of interface, in X and Z directions, of the j-th functional element of the building component, within the surface iterative modulus MS_j of the functional element; f_{ik} = frequency of i-th type of interface, in Y direction, of the k-th interface between functional elements of the building component, within the surface iterative modulus Ms_{kmax}.	
	Form adaptability	$F_i = 0.5$ between material and material or moulded element $F_i = 1$ between moulded elements
	Types of materials	$M_i = 0.5$ same nature $M_i = 1$ different nature
	Fabrication process	$E_i = 0.5$ interface carried out in factory $E_i = 1$ interface carried out in site

Table 4: Calculation of criteria for fabrication compliance

Calculation of criteria	
$I_j^{X,Z} = \sum_{i=1}^{N_j^{X,Z}} N_{ij}^{X,Z}$ (12)	N_{Xj}= total number of interfaces, in direction X, within the surface iterative modulus MS_j of j-th functional element; N_{Zj}= total number of interfaces, in direction Z, within the surface iterative modulus MS_j of j-th functional element;
$I_j^Y = \sum_{k=1}^{N_k^Y} N_{ik}^Y$ 13)	N_{Yk}= total number of interfaces, in direction Y, within the surface iterative modulus Ms_{kmax} of k-th interface between functional elements of the building component.
$I^{X,Z} = \sum_{j=1}^{N_E} I_j^{X,Z}$ (14); $I^Y = \sum_{k=1}^{N_k} I_k^Y$ (15)	N_E = number of functional elements of the building component; N_K= number of interfaces, in direction Y, between functional elements of the building component.

2.3 Inherent compliance

The propensity of a given building component to meet functional requirements in regards to its inherent compliance is conducted on designed component. It is based on an estimate of the occurrence of specific physical phenomena, i.e. inherent characteristics of the materials to change dimension given changes in temperature or humidity. These local changes in the materials in turn bring about dimensional change on both the functional element and likewise the component over their service life. The objective model of the building component is evaluated according to Tables 4 to 9.

Table 5: Calculation of criteria for inherent compliance

Criteria Class	Criteria	Calculation of criteria	Index
Inherent variability, according to direction X	Inherent thermal variability, X direction (P_{TX}) Inherent humidity variability, X direction (P_{HX})	$I_{T,HX} = \frac{\sum_{i=1}^{N} P_{TXj} . P_{HXj}}{N}$ (15) N = number of functional elements of the building component;	I_{TX} I_{HX}
Inherent variability, Z direction	Inherent thermal variability, Z direction (P_{TZ}) Inherent humidity variability, Z direction (P_{Hz})	$I_{T,HZ} = \frac{\sum_{i=1}^{N} P_{TZj} . P_{HZj}}{N}$ (16)	I_{TZ} I_{HZ}
Inherent variability Y direction	Inherent thermal variability Y direction (P_{UY}) Inherent humidity variability Y direction (P_{HY})	$I_{T,HY} = \frac{\sum_{i=1}^{N} P_{TYj} . P_{HYj}}{N}$ (17)	I_{TY} I_{HY}
Inherent thermal variability		$I_T = I_{TX} + I_{TZ} + I_{TY}$ (18)	I_T
Inherent humidity variability		$I_H = I_{HX} + I_{HZ} + I_{HY}$ (19)	I_H

Table 6: Calculation of criteria for inherent compliance

Calculation criteria	
Inherent thermal, humidity variability according to direction X $P_{TXj,HXj} = \frac{\sum_{i=1}^{k}(\alpha_j.\beta_j) \cdot l_j \cdot p_j}{ML_j}$ (20)	k= number of elements constituting the functional element in direction X, within surface iterating modulus; α_i, β_i = coefficients of thermal, humidity variability of the i-th element present in the j-th functional element of the building component; l_j = length of the i-th element expressed in meters; P_i = correction factor for the i-th element, function of the modulus of elasticity of the element and of the arithmetical mean of the elastic moduli of the relative functional connections, as well as of the parameter of the moment of inertia of section YZ of the i-th element and of the connections types [yz3/12]; ML_j = value expressed in meters of the linear iterating modulus according to X of the j-th functional element
Inherent thermal, humidity variability according to direction Z $P_{TZj,HZj} = \frac{\sum_{i=1}^{r}(\alpha_j.\beta_j) \cdot h_j \cdot p_j}{MH_j}$ (21)	r = number of elements constituting the functional element in direction Z, within surface iterating modulus; α_i, β_i = coefficients of thermal, humidity, shrinkage variability of the i-th element present in the j-th functional element of the building component; h_j = height of the i-th element expressed in meters; P_j = correction factor for the i-th element, function of the modulus of elasticity of the element and of the arithmetical mean of the elastic moduli of the relative functional connections, as well as of the parameter of the moment of inertia of section YX of the i-th element and of the connections types [yx3/12]; MHj = value expressed in meters of the linear iterating modulus according to Z of the j-th functional element
Inherent thermal, humidity variability according to direction Y $P_{T,HY} = \lvert\alpha_{i1}.\beta_{i1} - \alpha_{i2}.\beta_{i2}\rvert \cdot p_j$ (22)	e_{i1}, e_{i2} = elements with highest superficial development on surface XZ present respectively in the two functional elements at the i-th interface of the building component $\alpha_{i1}.\beta_{i1}$, $\alpha_{i2}.\beta_{i2}$, = coefficients of thermal, humidity, variability of the two functional elements e_{i1}, e_{i2} P_i = correction factor for the i-th interface, function of the elastic moduli of e_{i1} and e_{i2}, of the moments of inertia in sections XZ of the two elements e_{i1} and e_{i2}, as well as of the connection type of the j-th interface

Table 7: Correction factors and modulus of elasticity classes

P_c in relation to different types of functional connections for X and Z directions

Types of functional connection	Correction factors P_c
Location	1
Closed	0.8
Flowing	0.4
Open	0.2

Modulus of elasticity (E) classes

Class	Range E (kN/mm^2)
A	$E > 80$ kN/mm^2
B	10 kN/mm$^2 < E\ 80$ kN/mm^2
C	$E\ 10$ kN/mm^2

Table 8: Correction factors P_E and P_I

P_E in relation to different classes of modulus of elasticity (MOE) for elements and mean value for two functional connections (directions X and Z)

Classes MOE: element	Classes MOE: connections	P_E
A	A	1
A	B	0.9
A	C	0.8
B	A	0.7
B	B	0.6
B	C	0.5
C	A	0.4
C	B	0.3
C	C	0.2

P_I in relation to different classes of moment of inertia (J) of element sections YZ, YX, XZ

Ranges for moment of inertia J (cm^4/m)	P_I
$10^7 < J$	1.00
$10^6 < J\ 10^7$	0.87
$10^5 < J\ 10^6$	0.73
$10^4 < J\ 10^5$	0.60
$10^3 < J\ 10^4$	0.47
$10^2 < J\ 10^3$	0.33
$J\ 10^2$	0.20

Table 9: Correction factors Y-component direction

P_S for different types of functional connections

Types of functional connection	P_S
Continuous	1.0
Linear	0.7
By location	0.6
Open	0.2

P_E for different classes of modulus of elasticity (MOE) and interface elements e_{i1} and e_{i2}

Class MOE, element e_{i1}	Class MOE, element e_{i2}	P_E
A	A	1.00
A	B	0.84
A	C	0.68
B	B	0.52
B	C	0.36
C	C	0.20

P_{Iei1}, P_{Iei2} for different classes of Moment of inertia and j-th interface

Ranges: MOE J (cm^4)	P_{Iei1} or P_{Iei2}
$10^7 < J$	1.00
$10^6 < J\ 10^7$	0.87
$10^5 < J\ 10^6$	0.73
$10^4 < J\ 10^5$	0.60
$10^3 < J\ 10^4$	0.47
$10^2 < J\ 10^3$	0.33
$J\ 10^2$	0.20

2.4 Compatibility compliance

The evaluation of the compatibility compliance is undertaken on the designed component and is based on an estimate of possible failures due to chemical and physical incompatibilities between two sets of adjacent functional elements within a given component. The design of the component is analysed and evaluated according to the criteria provided in Table 10:

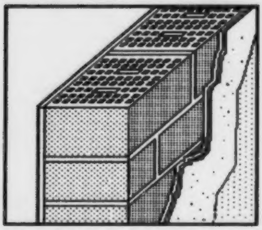
Table10: Calculation criteria for compatibility compliance

Criteria Class	Calculation of criteria	Index
Physical incompatibility	$I_{phij} = \sum_{i=1}^{n_i} P_{phji}$ (23) n_i = number of physical incompatibilities in the jth interface P_{ji} = correction factor relative to the physical incompatibility in the jth interface $Pph_i = \frac{\sum_{i=1}^{N_i} I_{phij}}{N_i}$ (24) N_i = Total number of interfaces between different materials within the building component	I_{ph}
Chemical incompatibility	$I_{chij} = \sum_{i=1}^{n_i} P_{chji}$ (25) n_i = number of chemical incompatibilities in the jth interface P_{ji} = correction factor relative to the chemical incompatibility in the jth interface $Pch_i = \frac{\sum_{i=1}^{N_i} I_{chij}}{N_i}$ (26) N_i = Total number of interfaces between different materials within the building component	I_{ch}

3 Example of functional compliance analysis

The proposed functional compliance analysis method has been applied to a set of two non load-bearing external wall components. The building components that scored the maximum and minimum values for "global" compliance are given in Figures 1 and 2 respectively. Provided as well, are the specific values of functional, fabrication, inherent and compatibility compliance for each building component.

Alveolar Hollow Bricks

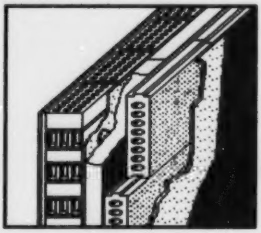


Component of the set of non load-bearing vertical perimeter walls

Compliance Scores	
Global	93
Functional	95
Fabrication	86
Inherent	96
Compatibility	95

Fig. 1: Building component with highest global compliance score

Semi-Hollow Bricks and Gypsum Blocks



Component of the set of non load-bearing vertical perimeter walls

Compliance Scores	
Global	77
Functional	91
Executive	32
Inherent	90
Critical	96

Fig. 2: Building component with lowest global compliance score

4 References

Maggi, P.N., Gottfried, A., Morra, L., (1992) *Qualità tecnologica dei prodotti complessi per l'edilizia*, Quaderno Diset n. 1, Esculapio, Bologna.

Maggi, P.N., Daniotti, B., (1993) *Valutazione della qualità tecnologica caratteristica dei prodotti complessi per l'edilizia*, Quaderno Diset n. 1, Esculapio, Bologna.

Maggi, P.N., Rejna, M., Ravetta, F., (1998) *Functional analysis as a method to design new building component*, CIB World Building Congress, Gavle, Sweden.

Rejna, M., (1995) Valutazione della qualità tecnologica utile dei prodotti complessi per l'edilizia, Quaderno Diset n. 4, Esculapio, Bologna.

139 PHENOMENOLOGICAL MODEL OF BUILDING MATERIAL STRAIN

O.V. Sinyayev

Cherepovets State University, Cherepovets, Russia

Abstract

The model uniting phenomenological models of Hooke, Newton, Maxwell, Kelvin-Voigt, Maxwell-Thomson, Ziner which are asymptotic with respect to the proposed model have been constructed. Mechanical condition of the material is characterized by the value of dimensionless parameters α and β. Parameter α is determined by the ratio of long-term and instantaneous elastic modulus or by the ratio of relaxation and retardation time. Parameter β is determined by the value of the residual strain. It is shown that the proposed model makes it possible to describe phenomena of elasticity, plasticity, viscoelasticity and viscoelastoplasticity.

Keywords: Linear standard body, elasticity, plasticity, viscoelasticity, creep, viscoelastoplasticity, after-effect, symbolic modulus

1 Introduction

The majority of construction materials demonstrate strongly pronounced dependence of mechanical properties on temperature and time factors. The ordinary manifestation of this phenomenon is the material creep under the load.

When testing a sample under constant load its dimensions are changed according to the law determined by physical and chemical processes in the material which are accelerated or retarded depending on availability or absence of the field of stress and temperature value. When the load is removed the sample shortens instantly and during some period of time it shortens according to the law different from the law of the strain growing.

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It should be noted that the actual instant strain is displacement under action of rapidly increasing stress during its growth. This time is considered to be greatly less than the time we are examining.

Usually the regularity of the strain tensor formation in loaded and unloaded state is determined using the description form ,of Boltzmann-Volterra integral equation. (Malmeister et al. 1972):

$$\varepsilon'_{\alpha\beta} = a_{\alpha\beta\gamma\delta..\theta\varphi}\sigma_{\gamma\delta...}\sigma_{\theta\varphi} + \int_0^t K_{\alpha\beta\gamma\delta..\theta\varphi}(t,\tau)\sigma_{\gamma\delta}(\tau)..\sigma_{\theta\varphi}(\tau)d\tau \; ; \tag{1}$$

$$\varepsilon''_{\alpha\beta} = \dot{\varepsilon}_{\alpha\beta} + \hat{a}_{\alpha\beta\gamma\delta..\theta\varphi}\,\sigma_{\gamma\delta...}\sigma_{\theta\varphi} + \int_0^t \hat{K}_{\alpha\beta\gamma\delta..\theta\varphi}(t,\tau)\sigma_{\gamma\delta}(\tau)..\sigma_{\theta\varphi}(\tau)d\tau \, ,$$

where K and $\hat{K}$ are nuclear functions.

In general case it is convenient to describe nuclear function as Yu.N.Rabotnov's rational and exponential function as varying of parameters α and β makes it possible to obtain a wide range of known nuclei.

Service life of constructional structures exceeds the time of the intensive creep of the material therefore engineers are interesting mainly in its behavior when values of the time are high, In this case it is advisable to approximate the creep nucleus in the form of an exponent.

2 Phenomenological model of creep and after- effect

The equation of state which describes properties of the Maxwell-Thomson model (TM model) is often used in case of the fixed temperature and non-aging material

$$\sigma + n\dot{\sigma} = \mathrm{E}(n\dot{\varepsilon} + \alpha\varepsilon)\text{, where } \alpha = n/\eta_\varepsilon = \mathrm{H}/\mathrm{E}, \tag{2}$$

$n_\varepsilon = En/H$; n_ε - lag time (retardation time); n - relaxation time of stress; H - long-term elastic modules; E - instantaneous elastic module. Taking into account that the creep phenomena appears if E>H one can receive the range of the parameter α change: $0<\alpha<1$.

Let's consider the known differential equation of Ziner model of a standard linear solid body (Ziner 1954)

$$\sigma + n\dot{\sigma} = \mathrm{M}(\varepsilon + \eta_\varepsilon\dot{\varepsilon}), \tag{3}$$

where M - conventional elastic modulus. The rest designations are the same as for the equation (2).

Let's introduce relation

$$\beta = M/H = M/(\alpha E) \tag{4}$$

into equation (3), where α is determined by the formula (2).

As a result we receive from (3)

$$\sigma + n\dot{\sigma} = \beta\, E(n\varepsilon + \alpha\dot{\varepsilon}). \tag{5}$$

Assume (Sinyayev 1994) that strains of the direct creep (loading) are determined by equation (5) and strains of the reverse creep (unloading) - by equation (2). In a first approximation it corresponds to experimental data obtained for a wide range of constructional materials.

In case of an arbitrary law of the stress changing in time we receive from (5)

$$\varepsilon(t) = \left(\varepsilon_0 - \frac{\sigma_0}{\beta E}\right) e^{-\frac{\alpha t}{n}} + \frac{\sigma(t)}{\beta E} + \frac{1-\alpha}{\beta E n}\int_0^t \sigma(\tau) e^{-\frac{\alpha(t-\tau)}{n}} d\tau . \tag{6}$$

When $\varepsilon_0 = \varepsilon_e = \sigma_0 / E$ and $\sigma = const$ the formula (6) is

$$\varepsilon(t) = \frac{\sigma_0}{\alpha\beta E}\left[1 + (\alpha\beta - 1) e^{\frac{-\alpha t}{n}}\right] \tag{7}$$

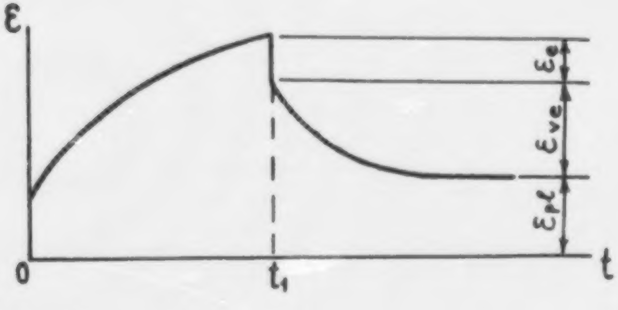


Fig.1: Curve of direct and reverse creep

At the moment of unloading $t = t_1$ the value of strain $\varepsilon(t_1)$ (fig.1) is determined by formula (7).

Due to the instantaneous decrease of stress by the value σ_0 at the same moment of time t_1 the strain begins to change by the law

$$\varepsilon(t_{aef}) = \varepsilon(t - t_1) = \frac{\sigma_0}{\alpha\beta E}\left[1 + (\alpha\beta - 1) e^{\frac{-\alpha(t-t_1)}{n}}\right] \tag{8}$$

The law (8) is the solution of equation (2) on the strain when $\sigma = const$. The strain of the reverse creep is determined by the formula

$$\begin{aligned} \varepsilon_{aef}(t - t_1) &= \varepsilon(t_1) - \varepsilon(t_{aef}) = \\ &= \frac{\sigma_0}{\alpha\beta E}\left[1 - \beta + (\alpha\beta - 1) e^{\frac{-\alpha t_1}{n}} - \beta(\alpha - 1) e^{\frac{-\alpha(t-t)_1}{n}}\right] \end{aligned} \tag{9}$$

In time the strain after unloading tends to the value $\varepsilon(\infty) = \sigma_0(1-\beta)/(\alpha\beta E)$, i.e. in general the strain process is irreversible. When $\beta = 1$ the strain after unloading tends to zero and the strain process becomes reversible.

Fig.2,a shows diagram of direct creep $\varepsilon(\theta)/\varepsilon_e{+}\theta = t/n$ in relative coordinates obtained for some values α and β by formula (7).

Fig.2,b shows the diagram of reverse creep $\varepsilon_{unl}(\theta)/\varepsilon_e{+}\theta_{unl} = (t - t_1)/n$ in relative coordinates obtained for some values α and β by formula (9).

If sums of strains are equal in case of direct and reverse creep we obtain

$$\beta = 1/(1+\alpha\varepsilon_{pl}/\varepsilon_e). \qquad (10)$$

where ε_{pl} - plastic (residual) strain; ε_e - instantaneous elastic strain. Diagrams of β on $\varepsilon_{pl}/\varepsilon_e$ when α is fixed are given in Fig.3.

3 Experimental test of the model

Numerical value of the parameter β should be determined with the help of standardized maximum permissible values $[\varepsilon_{pl}/\varepsilon_e]$. For example, the author of paper (.Kozlov 1966) considers that one should assume permissible strains as the main criterion when determining the durability and reliability of polymer structures. In this case it is important to assign maximum permissible strain and the time of its development without disturbance of the structure fitness for work. Similar approach is used for standards in force of steel structures when calculating their plastic stage of operation. In the most general case $\beta = \beta(\sigma;t)$.

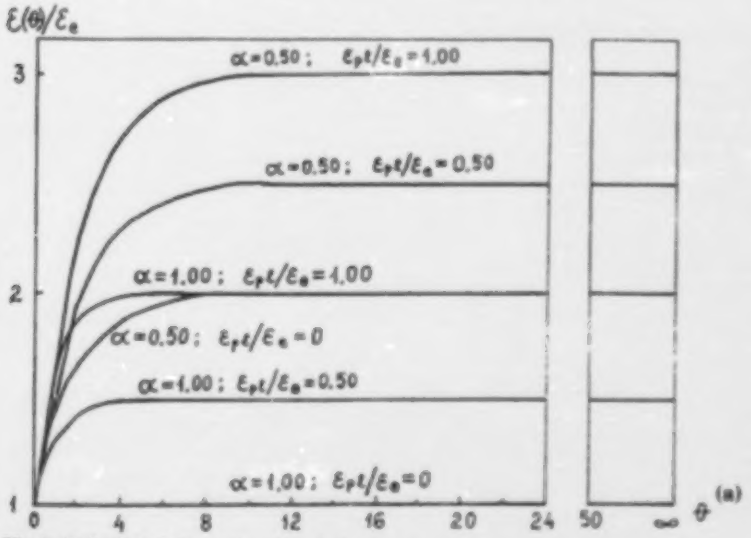
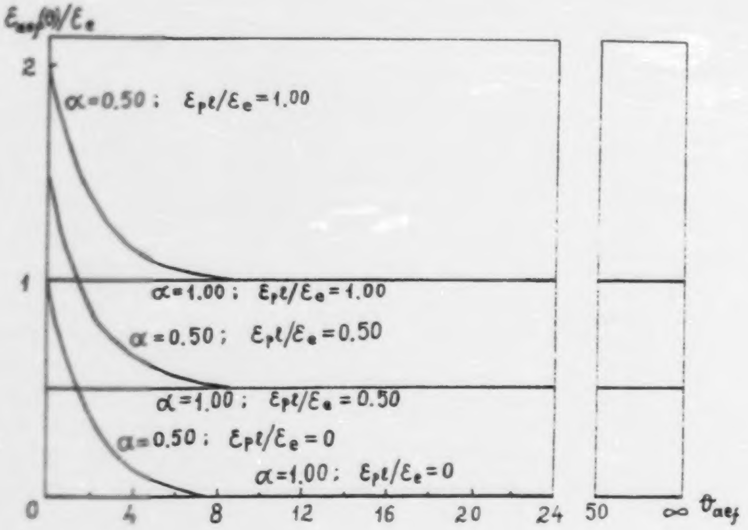


Fig.2: Calculated dependencies of direct (a) and reverse (b) creep



(b)

Fig. 2 (cont'd): Calculated dependencies of direct (a) and reverse (b) creep

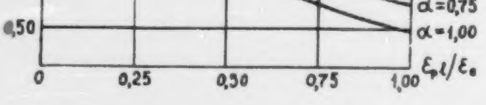


Fig.3: Calculated dependencies of parameter β on ratio $\varepsilon_{pl}/\varepsilon_e$ when values α are fixed

Fig.4 shows experimental (Kalinnikov 1965) and calculated by formula (7) diagrams of polyethyleneterephthalate film samples. Physical constants and rheological factors included into formula (7) are obtained as a result of processing of experimental curves (Kalinnikov 1965) and they take the following values:

$\alpha = 0.67$; $E=3600$ MPa; $n=14.5$ hr; when $\sigma=20$ MPa; $\beta=0.90$ and $\varepsilon_{pl}/\varepsilon_e=0.16$ when $\sigma_0=32$; $\beta=0.79$ and $\varepsilon_{pl}/\varepsilon_e=0.39$; when $\sigma_0=38$ MPa; $\beta=0.78$ and $\varepsilon_{pl}/\varepsilon_e=0.42$ when $\sigma_0=62$ MPa; $\beta=0.77$ and $\varepsilon_{pl}/\varepsilon_e=0.44$.

4 Conclusion

The best correspondence of the theoretical creep curves to the experimental data is observed at the final stage of the strain process. It is explained by the fact that parameter β is assumed to be independent of time. Such assumption simplifies the initial differential equation (5) and permits to calculate the wide range of constructional structures with sufficient accuracy for practical purposes. To provide coincidence of theoretical creep curves with experimental data in the initial stage of strain it is necessary to introduce the empirical dependence of parameter β on time into the equation (5). The initial differential equation becomes somewhat complicated but it remains linear with respect to stress and strain. Having introduced empirical dependence of parameters β on strain into the equation (5) we obtain nonlinear differential equation. So, the suggested mathematical model (5) describes adequately mechanical states of the material (tab.) in the wide range of stresses (temperatures): elasticity ($\alpha=1$; $\beta=1$, $\varepsilon_{pl}=0$), plasticity ($\alpha=1$; $0<\beta<1$, $\varepsilon_{pl}\neq 0$, $\varepsilon=\varepsilon_y$, where ε_y - strain, which corresponds to the yield limit σ_y), viscoelasticity ($0<\alpha<1$; $\beta=1$, $\varepsilon_{pl}=0$) and viscoelastoplasticity ($0<\alpha<1$; $0<\beta<1$, $\varepsilon_{pl}\neq 0$).

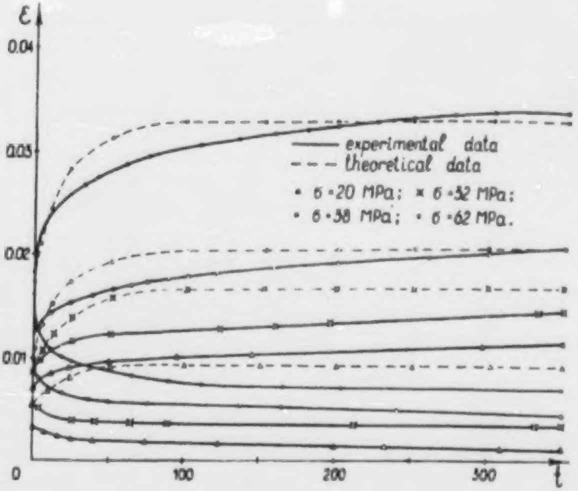


Fig. 4: Curves of direct and reverse creep of polyethyleneterephthalate

Table 1: Development of phenomenological models describing the material viscoelastoplasticicity properties

No.	Type of equation		Author
1	$\sigma + \dot{\sigma} n = \beta E(n\dot{\varepsilon} + \alpha\varepsilon)$		O.V.Sinyayev, 1993
	α=H/E	β=M/H=M/(αβ)	
	0≤α≤1	0 ≤β≤1/α	
	E – instantaneous modulus of elasticity; H – long-time modulus of elasticity; M – conventional modulus of elasticity		
2	α=β		V.P.Kotsegubov, 1974
	$\sigma + \dot{\sigma} n = \alpha E(n\dot{\varepsilon} + \alpha\varepsilon)$		
	$\alpha = n_+/n_- = \dot{\varepsilon}_+/\dot{\varepsilon}_- = \eta_+/\eta_-$		
	n_+, n_- - times of relaxation of relative strains with direct and reverse creep; $\dot{\varepsilon}_+$, $\dot{\varepsilon}_-$ - speeds of relative strains with direct and reverse creep; η_+, η_- - coefficients of viscosity with direct and reverse creep.		
3	0 ≤α≤1	β = 1	Maxwell, Thomson
	$\sigma + \sigma n = E(n\varepsilon + \alpha\varepsilon)$		
4	α = 0	β = 1	Maxwell
	$\sigma + \sigma n = En\varepsilon$		
	α= 0	0 ≤β≤1/α	Maxwell's model with modulus β
	$\sigma + \sigma n = \beta En \varepsilon$		
	0 ≤α≤1	β= 0	relaxation of stress by Maxwell's model
	$\sigma + \sigma n = 0$		
5	0 ≤α≤1	β→∞	elastic recovery by Kelvin-Voigt model
	$E(n\varepsilon + \alpha\varepsilon) = 0$		
6	α = 1	β = 1	Hooke
	$\sigma + \sigma n = E(n\varepsilon + \varepsilon)$		

5 Reference

Malmeister A.K., Tamuzh V.P., Teters G.A. (1972) *Strength of rigid polymer materials*, Zinatne, Riga.

Ziner K. (1954) Elasticity and non-elasticity of metals, in *Elasticity and non-elasticity of metals* (ed. S.V.Vonsovski. FL, Moscow, p.p. 9-168).

Kozlov P.M. (1966) *Application of polymer materials in structures under load*, Khimia, Moscow.

Kalinnikov A.E. (1965) Creep and aftereffect of polyethyleneterephthalate film in conditions of single-axis stressed state. *Mechanics of Polymers*. No.2. p.p. 59-63.

Sinyayev O.V. (1994) Phenomenological model of building materials deformation. Construction. No.2. p.p. 29-33.

140 SERVICE LIFE PREDICTION AND ECONOMIC ASSESSMENT OF PARKING GARAGE OPTIONS

A. Wiseman and B.R. Kyle

Public Works and Government Services Canada, Hull, QC, Canada

Abstract

Parking garage configurations in Canada present one of the most adverse climates for reinforced concrete. The historic excessive use of de-icing salts on North American roadways has, by exposing the structural elements to a saline solution at a heightened ambient temperature, enhanced the rate of deterioration in parking structures. Various rehabilitation options may be considered for different circumstances, from simple stripping and repair of affected areas, to the chloride extraction or re-alkalinisation of structural elements, and ultimately to demolition and reconstruction. The emergence of "innovative" materials and methods promising extended or altered in-service performance has left designers without a clear view of the relative benefits of the more traditional approaches. This paper examines the degradation models available for service life prediction of parking structures, in particular methods to evaluate systems using innovative materials. This paper also presents the projected service life extensions provided by design, rehabilitation and material options and evaluates the life-cycle economics of their application to a specific parking garage scenario. The assessment considers the total life-cycle costs, including capital, operations and maintenance costs, re-capitalisation as well as the projected rehabilitation costs for each scenario.

Keywords: service life prediction, total life-cycle costs, life-cycle economics, parking garages, reinforced concrete, deterioration, fibre reinforced polymers

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1 Introduction

Over the past number of years, the Technology Directorate of Public Works and Government Services Canada's (PWGSC) has been investigating both conventional construction techniques and alternate technologies. An emphasis is placed on life cycle costs for projects including the frequency and cost of future maintenance and replacement work, as opposed to considering only the initial capital costs. PWGSC has produced a durability guideline on the design, construction, maintenance and repair of parking structures using conventional methods (PWGSC 1994).

Preliminary analysis shows that selective use of non-conventional materials may increase the initial capital project costs. However, this cost increase can be offset by extended service life, and reduced operation and maintenance costs. If "disruption costs" are added to maintenance costs the case becomes even stronger. Costs such as lost revenue can be easily calculated, while soft costs such as client disruption are more difficult to quantify.

This paper examines the degradation models available for service life prediction of parking structures, in particular methods to evaluate systems using newer materials. This paper also presents the projected service life extensions provided by design and material options and evaluates the life-cycle economics of their application to a specific parking garage scenario. The assessment considers the total life-cycle costs, including capital, operations and maintenance costs, as well as the projected re-capitalisation costs for each scenario.

2 Description of "Case" structure - Laurier-Taché garage

The Laurier-Taché multi level garage was built in 1973 and is located under the intersection of Maisonneuve Boulevard, Taché Boulevard, and Laurier Avenue in Hull, Quebec. The structure consists of approximately 30,000m^2 of structural slabs. The 125mm± one-way slabs are supported at 3.9m intervals by post-tensioned beams for a total of 388 panels (Figure1). Slab replacement is being considered for this parking structure.

The Laurier-Taché garage lies partially beneath the Place du Portage complex. The Canadian design guidelines for enhanced building durability (CSA S478, 1995) recommends design service life categories. This structure is classified as a long life facility, requiring a minimum design service life of 50 years. With this taken into consideration, the design service life of the parking structure used for the remainder of this study is 50 years.

3 Retrofit and rehabilitation options

Conventional garage repair has consisted of the "chip and patch" method. The level of intervention was determined by budget, severity of spalling, delamination, chloride contamination and structural system. Other strategies include chloride extraction and re-alkalization, and cathodic protection. (Nocure 1996; PWGSC 1994). More recently, the use of a corrosion inhibitor in the concrete mix has been employed (CSA S413-95).

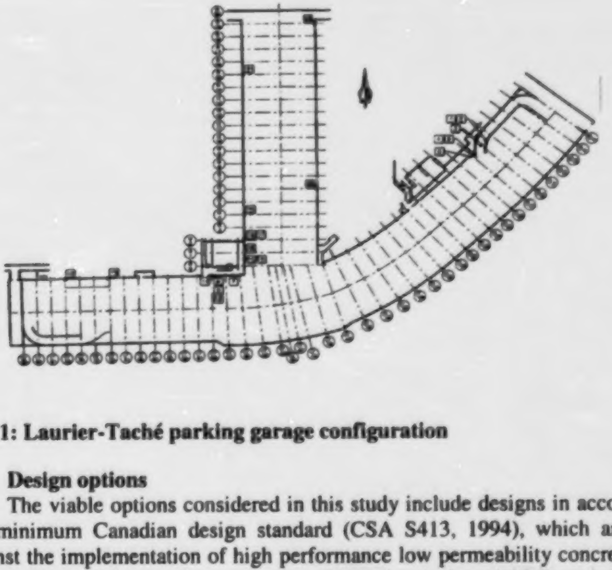

Fig. 1: Laurier-Taché parking garage configuration

3.1 Design options

The viable options considered in this study include designs in accordance with the minimum Canadian design standard (CSA S413, 1994), which are evaluated against the implementation of high performance low permeability concrete (HPC) as well as fibre-reinforce polymers, carbon (CFRP) and glass (GFRP) and stainless steel reinforcements. General design configurations of the options considered are presented in Table 1.

Table 1: Design options

Option	Concrete	Reinforcement	
		Top	Bottom
1-CSA S413	Conventional	epoxy coated	plain
2-HPC	High performance	epoxy coated	plain
3-Stainless	Conventional	stainless	plain
4-Carbon FRP	Conventional	CFRP	epoxy coated
5-Glass FRP	Conventional	GFRP	epoxy coated

3.1.1 Fibre reinforced polymers

Carbon and glass fibre reinforced polymer (CFRP, GFRP) rods are non-corrosive, lightweight, and have a high tensile strength. The initial mechanical properties of these products have been established, however gaps in information remain as to the long-term durability in the service environment. The emergence of FRPs has promised an extended service life by using materials inert to chloride ion attack. The bars are made of plastic resins reinforced with either glass or carbon and come in shapes made to resemble conventional reinforcing bars or mesh. The first products on the North American market, in the late 1980's, were reinforced with glass fibres. Testing found that the stressed glass reinforcement, in the alkaline environment of a concrete slab, experienced a reduction in tensile strength (Adimi 1998; Gerritse 1998; Benmokrane 1998). The carbon rods generally have very high tensile strength and low modulus of elasticity in comparison to steel. To date, we have not found any data on the long-term performance of FRP field installations.

3.1.2 Stainless steel

Another product that has proven to be highly resistant to corrosion is stainless steel reinforcing bars. The Institute for Research in Construction (IRC) at the National Research Council (NRC) has carried out corrosion testing on stainless steel bars (Gu 1997).

3.1.3 High performance concrete

The benefits of using supplemental cementious materials have been known for sometime (Carette and Malhotra 1983). The advantages potentially include: 1) reduced permeability; 2) increased strength; 3) reduced drying shrinkage and creep; 4) carbonation resistance; and 5) superior heat dissipation and crack control in mass pours. Recent studies have also evaluated the performance of steel reinforcement within these concretes (Gu et al. 1998). The successful use of high performance concrete is dependent upon an enhanced level of quality control and curing practices

Concrete permeability is a significant factor affecting the rate at which chloride ions migrate through concrete. CSA S413-94 states that low-permeable concrete shall mean a water/cementing materials ratio not exceeding 0.4 and an average coulomb rating not exceeding 1500 (CSA S413 1994). With the use of specialty mixes using supplementary cementing materials this can be reduced to 250 – 1000 coulombs (Aïtcin 1998).

4 Modelling of concrete deterioration, maintenance and service life

4.1 Background work

Considerable research work has been devoted to the evaluation of chloride ion threshold levels and corrosion in reinforced concrete (Tuutti 1982). The onset of corrosion and its progression may be significantly affected by the curing temperature, the cement composition, pore structure, the alkalinity of pore water and concrete as well as sorptivity and porosity.

The mechanisms of deterioration and the relative effectiveness of various garage protection mechanisms have been evaluated for Canadian exposure conditions (Litvan and Bickley 1987). Further works examined chloride ion attack in prestressed concrete parking garage decks and a finite difference method was used to predict the time dependent change of chloride concentration profile (Funahashi 1990).

4.2 Reliability-based methods

Deterministic approaches are the general rule in civil engineering design. The design parameters usually consist of selected factors of safety multiplied by expected loads. However, these loads are rarely known with certainty and are best treated as random variables. This variability calls for the implementation of probabilistic and statistical techniques. Such methods have provided the basic framework for the study of civil engineering reliability problems.

Thoft-Christensen and Sørensen (1987) proposed a reliability-based methodology to optimize inspection, maintenance and repair of structural systems. The model minimizes total inspection and repair costs while maintaining system reliability to an acceptable level.

In last decade researchers have begun applying reliability theory to repair, maintenance and management of important structures such as nuclear power stations (Naus 1986; Clifton and Knab 1989). The analysis of time-dependent changes in system reliability has only been considered in the recent past (Mori and Ellingwood 1992).

First-order reliability methods have been proposed for the maintenance management of parking garages by monitoring the degree of deck delamination and spalling (Attwood et al. 1991). These models and techniques were successfully calibrated and benchmark tested on a PWGSC parking garage (Kyle, 1994). By expanding this work, analytical tools for the reliability-based prediction of remaining service life, considering time dependent changes in flexural strength, punching shear, deflection, as well as delamination, have been developed (Cheung and Kyle 1996).

4.3 Service life prediction

This study implements the PWGSC first order reliability-based analysis system to predict the service lives for traditionally reinforced concrete parking structures and extends these models to consider the implementation of high performance concrete (HPC), stainless steel reinforcement. The typical input distributions and parameters used in the service life prediction modelling are presented in Table 2.

Table 2: Input parameters for concrete deterioration modelling

Variable		Distribution	Parameters	
			Mean *	Standard Deviation *
Operating Temperature	(° C)	Uniform	16 to 18	-
Slab thickness	(mm)	Normal	175.0	8.75
Span	(m)	Normal	3.9	0.02
Water/Cementious Material		Normal	0.40	0.05
F_c	(MPa)	Normal	35.0	6.30
Reinforcement				
Diameter	(mm)	Normal	15.9	0.16
Spacing	(mm)	Normal	230.0	12.0
Concrete Cover	(mm)	Lognormal	40.0	17.5
F_y	(MPa)	Normal	400.0	36.0
E_{steel}	(MPa)	Normal	200E+03	0.007

The effect of a heightened maintenance program upon the life of the parking garage slab was examined by reducing the chloride ion load used in the simulations. The expected in-solution chloride ion load (topside of slab) in the Ottawa-Hull area is 8560 ppm for an average of 110 days per year (Kyle, 1994). The value used for simulation of increased cleaning program was conservatively set at 2500 ppm for the same duration. This translates to a weekly washdown schedule compared to the monthly cleaning currently undertaken. The effect of this reduced load, an extension of service life of 4 years (from 11 years to 15 years), is illustrated in the reliability-time plot, Figure 2, for an untreated slab with plain steel.

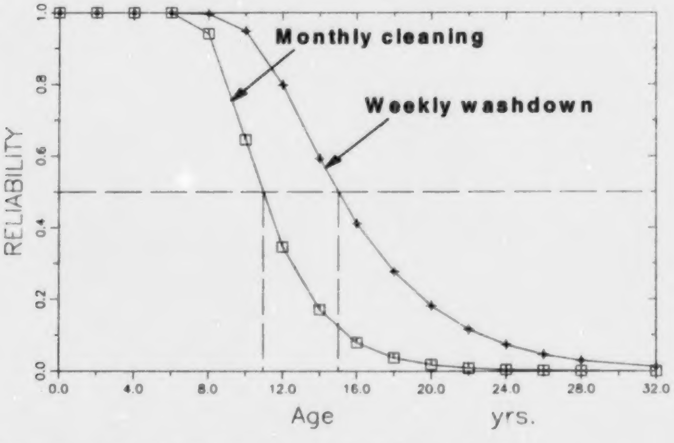

Fig. 2: The effect of reduced Cl^- load via increased maintenance

From recent experience with PWGSC parking garages, the structural capacity of the decks was rarely in question and in most instances the serviceability of the top surface was the identified reason for slab replacement and end of service life. This usually occurred when the deck was approximately 15% delaminated. In order to distinguish between the deterioration of the upper and lower mats of reinforcement the analysis examines serviceability and structural service life separately.

Design Options 1 through 3 (Table1) have ferrous-based top mats of reinforcement. For these options, the end of the "serviceability" service life is defined as the age when there is a 50% likelihood that surface delamination will be 15% or greater. The determination of the service life of the top reinforcement for Design Options 4 and 5 is not as clearly defined. Due to the "newness" of FRP use, the industry does not have a firm knowledge of the in-service durability. Therefore we have examined FRP from an economic viability viewpoint (see Section 6) based on benefit(revenue)-cost analysis.

Figure 3 presents the probability density functions (pdfs) of top mat performance for design options 1 to 3. They were derived from the reliability-time curves developed using the PWGSC parking garage deterioration model.

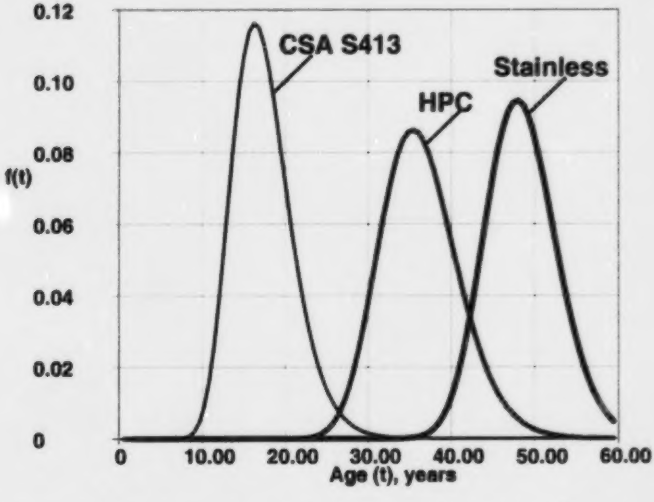

Fig. 3: Probability density functions of service life for options 1 to 3

The examination of structural capacity considers the change of section and resistance with time (Cheung and Kyle 1996). For purposes of this study, end of structural life is defined as the age when there is a 50% probability of having inadequate capacity.

The analysis of these systems and chloride loading found that the predicted structural life was in excess of one hundred years and that in all cases the serviceability was the factor governing service life.

End of service life is defined as the age at which the system reliability, $\boldsymbol{R}$, is 50%. Table 3 presents the predicted service lives for design options 1 to 3 as well as the statistical data for the fitted curves of Figure 3.

Table 3: Predicted service lives

Design Option	Service Life (Age @ R=0.5) SL	Fitted Distribution	Parameters	
			Mean •	Standard Deviation •
1-CSA S413	17	Lognormal	2.83	.21
2-HPC	35	Lognormal	3.58	.13
3-Stainless	48	Lognormal	3.88	.09

5 Analysis of Life-cycle economic performance

Once the technical performance and corresponding service life of the parking deck options have been projected, it is necessary to assess and compare their economical performance. Life cycle cost (LCC) analysis examines the economic performance of a project or alternatives over a designated study period, and includes all incurred costs including construction/installation, operations, maintenance, repair, replacement, and disposal.

5.1 Methodology

Using the standardized procedures for LCC analysis (ASTM E 917,1994) the potential economic advantages of strategies with differing initial capital costs and projected future costs (i.e., operations, maintenance, repair and replacement) may be compared. The LCC in present-value terms (LCC_{pv}) is computed as follows:

$$\mathrm{LCC}_{pv} = \sum_{t=0}^{N} \mathrm{C}_t/(1+\mathrm{i})^t \qquad (1)$$

where C_t = sum of all relevant costs occurring in year t; N = length of study period, years, and i = discount rate.

If this approach is extended to examine potential revenue, the life-cycle benefits may be expressed as:

$$\mathrm{LCB}_{pv} = \sum_{t=0}^{N} \mathrm{B}_t/(1+\mathrm{i})^t \qquad (2)$$

where, N, and i are as above and B_t = sum of revenue occurring in year t. Potential revenues include income from parking space rental and expected residual, or resale, value at the end of study period (ie. at t=N).

A combined examination of revenue and cost streams, yields an indicator of, the life-cycle economic performance, expressed as follows:

$$LCE_{pv} = \sum_{t=0}^{N} (B_t - C_t)/(1+i)^t \qquad (3)$$

Life-cycle costing does not permit a comparison of financial performance of options if differing design lives (depreciation periods) are evaluated. For this reason the LCE will be used in conjunction with revenue-to-cost ratios in order to evaluate the relative investment effectiveness of the design options.

5.2 Assumptions and input parameters

The following assessment uses a study period, N, equal to the parking garage design life of 50 years. The discount rate, i, is set to 3.5%.

The costs of demolition, construction and rehabilitative maintenance used in this assessment use the most recent estimates for this project and have been adjusted to reflect the material and placement costs for each of the design options. Operation and maintenance costs have been estimated, as per the BOMA norm (BOMA 1994), to be one-eight of the capital cost of the slab with operations being approximately 90% of that budget. Based upon PWGSC experience, at approximately five year intervals 25% of the original capital cost of the membrane must be reinvested to maintain its integrity. All of the predicted service lives are shorter than the design life (Table 3) and re-capitalisation is required to prolong service. At the end of their respective service lives, the design options are restored to initial performance at a cost representing the demolition and through-slab reconstruction of half the parking garage surface area.

The 1728 parking stalls of the garage are generally 100% occupied or rented. The parking rate is around $100 per space per month, yielding a revenue of approximately $2M per year. Rehabilitation of the membrane or slab reduces the parking rental revenue in proportion with the area taken out of service. The residual value at the end of the study period is calculated as the product of the predicted reliability of performance, the fraction of remaining service life (**SLF**) and the initial capital cost of slab construction. Table 4 summarizes the cost and revenue flows for design options 1 to 3.

Table 4: Costs and revenue flows

Costs		CSA S413	HPC	Stainless
		Design Options		
Capital @ t=0 years	($K)	6742	7292	7949
Annual operations	($K)	758	758	758
Maintenance	Annual ($K)	84	84	84
	5 year cycle ($K)	118	118	118
Re-capitalisation	($K)	3863	4138	
	Occurrence (year)	17 & 34	35	48
Revenue				
Parking rental ($K)	In re-capitalisation year	1037	1037	1037
	In maint'nce cycle year	1555	1555	1555
	In typical year	2074	2074	2074
Residual @ t=50 years	$R_{t=50}$	.58	1.0	1.0
	SLF	0/17	20/35	48/48
	($K)	230	4167	7618

5.3 Predicted economic performance

The LCCs, LCEs and Revenue/Cost ratios for design options 1 to 3, at a 50 year design life, are presented in Table 5 and illustrated in Figure 4.

Table 5: Life-cylce cost and economic indicators for 50 year design life

Design Option	LCC_{pv} $K	LCE_{pv} $K	Revenue / Cost
1 CSA S413	30375	15136	1.50
2 HPC	28815	18145	1.63
3 Stainless	29088	18540	1.64

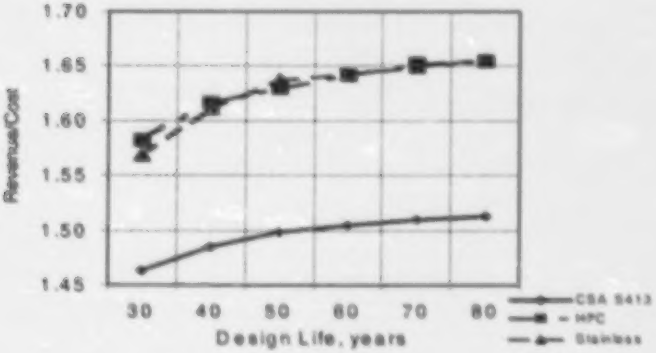

Fig. 4: Variation in revenue-to-cost with design life

6 Discussion

The predicted service life of design options 1,2 and 3 are 17, 35 and 48 years respectively. Therefore, it is not possible to achieve even a 25 year service life without a major intervention if the standard requirements of S413 are followed. In order to achieve a 50 year design life option 1 would require 2 major interventions (at year 17 and year 34).

The HPC and stainless steel options yield considerably higher LCEs than the CSA S413 option and have approximately the same revenue-to-cost ratios for a 50 year design life. An evaluation of the revenue-to-cost ratios for design options 1 to 3, across a range of design lives, indicate that Options 2 and 3 provide approximately the same economic performance and are consistently higher than Option 1, the CSA S413 compliant design. These findings are presented in Figure 4.

In order for the FRP materials to be economically viable it was determined that the required service life of the CFRP rods is in the range of 50 to 55 years and for GFRP rods 30 to 35 years. These values are arrived at by using comparable values and assessments for LCE and revenue/cost ratios as HPC and stainless steel. They are compatible with the recommended values from CSA S478 Durability Guidelines for Buildings for design service life.

7 Conclusions

Service life predictions should be based upon the best technical knowledge available and attempt to consider the variability inherent in material properties; design loadings and construction practices. Reliability-based approaches satisfy this requirement. There must be a clear understanding of what is the desired service life and in what condition the structure will be in at the end of that service life.

Inadequate information exists to confidently predict the in-service performance of FRP reinforcement

Life-cycle economic performance (LCC, LCE and R/C) is a sound basis for comparison of viable system options

This study indicates that for this particular garage the slab design options 2 and 3 are consistently (across a range of design lives) the most economical.

8 Recommendations

Probabilistic techniques should be the standardised basis of asset service life prediction (future or remaining).

Economic evaluation techniques should be enhanced to take advantage of state-of-the-art developments by applying fuzzy logic and probabilistic assessment.

Further research should be undertaken to determine the field performance characteristics of FRP.

9 References

Adimi, R., (1998), Durability of FRP Reinforcements Under Tension-Tension Axial Cyclic Loading. *Proceedings of the 1st International Conference Durability of Fibre Reinforced Polymer (FRP) Composites for Construction*, pp. 635-641.

Aïtcin, P-C, (1998) *High-Performance Concrete*, University of Sherbrooke, P413, E_&FN SPON Imprint of Routledge, London, UK.

ASTM E 917-94, (1994) *Standard Practice for Measuring Life-Cycle Costs of Buildings and Building Systems.* American Society for Testing and Materials, Philadelphia, USA.

Attwood, D., Nessim, M.A., Ghoneim A., Cormeau, A. and Cheung, M.S., (1991) Application of reliability theory to in-service monitoring and maintenance of parking garages, *Canadian Journal of Civil Engineering*, Vol. 18, Number 5, pp781-788.

Benmokrane, B., (1998), Improvements of the Durability of FRP Reinforcements for Concrete Structures. *Proceedings of the 1st International Conference Durability of Fibre Reinforced Polymer (FRP) Composites for Construction*, pp. 571-585.

BOMA (1994) *Experience Exchange Report, Income/Expense Analysis for Office Buildings.* Building Owners & Managers Association International, Washington, DC, USA.

Canadian Standards Association (1994), CSA S413-94 *Parking Structures, Structures Design*, Canadian Standards Association, Etobicoke, Canada.

Canadian Standards Association, (1995) CSA S478-95 *Guideline on Durability in Buildings,* Canadian Standards Association, Etobicoke, Canada.

Cao, H.T. and Sirivivatnanon, V. (1991) Corrosion of Steel in Concrete with and without Silica Fume. *Cement and Concrete Research*, Vol. 21, pp. 316-324.

Carette, C.G. and Malhotra, V.M. (1983) Mechanical properties, durability and drying shrinkage of portland cement concrete incorporating silica fume. *ASTM J Cement, Concrete and Aggregates*, 5 (1) pp 3-13.

Cheung, M.S., and Kyle, B.R. (1996) Service life prediction of concrete structures by reliability analysis. *Construction and Building Materials*, Vol. 10, No. 1..

Clifton, J.R., and Knab, L.I, (1989) *Service life of concrete*, NUREG/CR-5466, National Bureau of Standards, U.S. Nuclear Regulatory Commission, Washington, D.C.

Funahashi, M., (1990) Predicting corrosion-free service life of a concrete structure in a chloride environment. *American Concrete Institute Materials Journal*, Vol. 87, No.6, pp.581-587

Gerritse, A., (1998), Assessment of Long Term Performance of FRP Bars in Concrete Structures, *Proceedings of the 1st International Conference Durability of Fibre Reinforced Polymer (FRP) Composites for Construction*, pp. 285-297.

Gu, P. (1997) *Corrosion Behaviour of Pre-stressed Stainless Steel Embedded in Chloride Contaminated Concrete*, National Research Council of Canada (NRCC), Ottawa, Canada.

Gu, P.; Beaudoin, J.J.; Zhang, M.H.; Malhotra, V.M. (1998) Performance of steel reinforcement in portland cement and high-volume fly ash concretes. *Proceedings of the CANMET/ACI International Symposium on Sustainable Development of the Cement and Concrete Industry*, October, pp. 467-489, Ottawa, Canada.

Kyle, B.R., (1994) *Field Trials and Benchmark Testing of Parking Garage Delamination Model*, Internal Report, Public Works and Government Services Canada, Ottawa, Canada.

Litvan, G., and Bickley, J., (1987) *Durability of Parking Structures: Analysis of Field Survey*, NRCC 28335, National Research Council of Canada, Institute for Research in Construction, Ottawa, Canada.

Mori, Y., and Ellingwood, B.R., (1992) Reliability-based service-life assessment of aging concrete structures. American Society of Civil Engineers, *Journal of Structural Engineering*, Vol. 119, No. 5, pp.1600-1621.

Naus, D.J., (1986) *Concrete component aging and its significance relative to life extension of nuclear power plant*, Report No. NUREG/CR-4652, Oak Ridge National Laboratory, Oak Ridge, Tenn.

Norcure (1996), *Is Salt Induced Corrosion Causing Problems with your Concrete Structures.*

Public Works & Government Services Canada (PWGSC), (1994) *Durability Guidelines for the Design, Construction, Maintenance and Repair of Parking Structures*, Doc. ST136e, PWGSC Documentation Centre, Ottawa, Canada.

Thoft-Christensen, P., and Sørensen, J.D., (1987) Optimal strategy for inspection and repair of structural systems. *Civil Engineering Systems*, Vol. 4, June, pp 94-100.

Tuutti, K. (1982) *Corrosion of steel in concrete*, CBI Research, Swedish Cement and Concrete Institute, No. 4, Stockholm, Sweden.

PART TEN

DEVELOPMENT AND USE OF GUIDES AND STANDARDS

141 LIFE CYCLE COSTING FOR A DESIGN LIFE STANDARD

A.F. Bennett,[1] I.C. Page,[1] K. Bourke,[2] S.N. Tucker,[3] and T. Nireki[4]

[1]Building Research Association of New Zealand, [2]Building Research Establishment UK, [3]CSIRO Building, Construction and Engineering, Australia, [4]Center for Better Living, Japan

Abstract

This paper reviews the progress made to date by Working Group 5 of ISO TC59 Building Construction SC14 "Design Life". Working Group 5 was set up to prepare a draft standard on life cycle costing (LCC) as a tool to be used when planning the design life of buildings. The LCC draft is intended to utilise the outputs generated by building practitioners using documents published as parts of ISO 15686 to enable an economic comparison of different planning/ design/ construction/ use/ disposal options so that the client's requirements can be fulfilled.

Keywords: Design life, life cycle costing, maintenance, buildings

1 Introduction

Attention has in recent times become directed at the concept of "design life" where materials performance is ideally matched to the intended function and life of the building. Durability and service life are still important aspects of design life, but are now part of a broader process which is intended to deliver the optimal building solution to the building owner and users, taking into account all relevant regulatory requirements.

Research on materials performance has often been stimulated by poor performance of building materials and components in service. Early failures of building materials and components cost national economies substantial sums of money due to the costs of maintenance and indirect costs associated with temporary (sometimes permanent) loss of functionality associated with maintenance activities.

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As a result, much of the research carried out on materials performance has focused on determining what causes the degradation of materials, how rapidly this occurs and how to slow or stop this degradation, but not how to best choose among options where known problems occur. CIB committee W80 has been active in this area for many years.

"Design life" is the title of a new subcommittee (SC14) of ISO TC59 "Building Construction". The members of the sub-committee had previously been active as WG9 in SC3. SC14 is working on six documents at the present time all related to design life as parts of ISO 15686. These are:

- general principles
- service life prediction
- auditing
- data formatting
- life cycle costing
- environmental sustainability.

The inclusion of life cycle costing (LCC) as a part of the SC14 working brief recognises that costs over the design life are often as important as form and functionality to the building owner.

2 Life cycle costing

2.1 General principles

Any analysis of an investment must include both initial and ongoing costs and returns over the period of the investment. Such an analysis allows investors to compare different options and decide which offers the best return for their particular circumstances over the lifetime of the investment. Construction works generally require a considerable initial capital investment and have lifetimes of 20-100+ years. As a result, building designs are often selected on the basis of the lowest initial acquisition cost, provided that certain functional and aesthetic parameters are met. This is particularly the case with design-build speculators who are often active when economic growth is bullish and intend to quickly on-sell the building, often before it is completed.

The overall concept of LCC is simple; all the costs associated with the design, construction, use, maintenance and disposal of the building are estimated or calculated over a defined period. This period may be the design life of the building or a shorter period. The costs are then adjusted to take into account interest rates and, in some cases, inflation, to bring the whole life cost over the defined period into today's currency. This then allows a comparison to be made between different design options.

3 ISO TC59 SC14 Draft standard

3.1 Work-to-date

A core group of ISO TC59 SC14 Working Group 5 met in Melbourne in November 1998 and produced the backbone of a draft titled "ISO 15686 Part 5 – Maintenance and life cycle costing". This document was circulated to the full working group prior to the end of 1998 for additional comment. Comment was incorporated into the draft and the revised draft circulated to the full SC14 sub-committee in March 1999. These comments were then considered at an SC14 WG5 meeting held prior to the 8th DBMC in Vancouver. The convenor of the Working Group is Kathryn Bourke from BRE.

3.2 Scope of the draft

The basic aim of the document is to facilitate a comparative assessment of the cost performance of construction works to be made over an agreed period of time. The term "cost performance" is important because the draft is not intended to be a complete investment tool. It is intended to enable comparisons of costs incurred by various design, construction, maintenance and disposal options. In some cases these options may have a direct effect on income associated with the building and may be included in the analysis. These are restricted to cases where maintenance operations (or lack of maintenance) or design options may directly affect future revenues such as rents or result in penalties.

It was recognised that as well as cost, investors in buildings will require a wider consideration of investment options, such as volume of space, location, type of investment (construction versus stocks or bonds etc.) and tax issues. However, the working group was of the view that a complete economic investment tool could not be realistically produced under the SC14 scope of work and that these issues should be left to a future ISO Standard or dealt with by existing economic analysis tools. It should be noted that documents have already been published which provide guidance on evaluating investment returns in buildings; documents such as ASTM E1185 (1993) and ASTM E1074 (1993). This draft is intended to cover construction works, which includes buildings, infrastructure items such as civil engineering works and building services.

3.3 Outline of draft

The draft includes the following sections. Note that the numbering does not refer to the numbering in the draft but is used for ease of reading in this paper.

3.3.1 Normative references

A mandatory ISO section covering other ISO standards which are referenced in the draft ISO 15686 - Part 5.

3.3.2 Definitions

Another mandatory section that explains terminology used in the draft and complements definitions included in other parts of ISO 15686.

3.3.3 Maintenance planning issues

Maintenance as a design life issue has not been extensively covered in the other parts of ISO 15686. Given the importance of maintenance in ensuring that the actual service life of a building matches the design life, this issue was originally scheduled to be covered in Part 5 along with LCC. The current draft recognises the role that maintenance plays in an LCC analysis, and that ideally the maintenance plan for a building is derived after the completion of the LCC. This is because the cost effectiveness of the different maintenance options can only be compared on a cost basis using LCC techniques.

Building maintenance is a sophisticated discipline in its own right and a great deal of information exists on this topic. One of the issues facing the Working Group is whether sufficient detail can be included in a Standard which covers both LCC and maintenance to do justice to both topics, or whether a separate part to ISO 15686 should be developed to cover aspects of maintenance outside those covered in LCC analyses. The activities of CIB W70 "Management, Maintenance and Modernisation of Building Facilities" are focused on maintenance and the input from this group will be sought on a formal basis by the time of the 8th DBMC.

3.3.4 Life cycle costing issues

The basic principles of LCC are set out, highlighting that LCC is driven by a client brief. LCC primarily sets out to provide information that will be used to assist in a decision making process. The phases of the life cycle are as follows:

- acquisition
- use and maintenance
- renewal and adaptation
- disposal.

The third phase may not exist but when it does, the use and maintenance phase commences again so that there are cyclic restoration and use steps which may repeat a number of times before the disposal phase occurs. Each of these phases may consist of a number of activities.

The *acquisition* phase covers the range of activities from the client's decision to seek useable "space or functionality" to the time at which occupation or use of the facility begins. It is this phase where much of the opportunity to influence whole life costs can be made. Decisions on location, size and type of structure, materials, services and energy sources can be altered much more readily before the structure is completed. This brings up the point that the full benefits of an LCC are realised only with new structures. However, LCC can have significant uses for existing buildings.

The *use and maintenance* phase is usually the longest phase in the life cycle of construction works yet is often the most neglected in the planning stages for major construction works. Choices made on the frequency and type of maintenance can have significant impacts on both the service life achieved and the costs incurred over the life of a structure. For existing buildings, LCC techniques can be used to evaluate the costs associated with various maintenance options for the whole or parts of a structure. This could include a review of the original maintenance schedule in light of the performance to-date or to evaluate alternative maintenance solutions resulting from unexpected failures.

The *renewal and adaptation* phase is essentially a partial repeat of all those activities in acquisition (plus some others) but at different times in the life of the works. *The use and maintenance* phase then begins again with different starting characteristics.

The *disposal* phase includes demolition, preparation for recycling and/or dumping as waste. Disposal may result in an income rather than a cost if the works have further potential use.

Life cycle costing can be carried out over the design life of the works, or over a finite period of the design life (the period of analysis). The technique is most commonly used for new works, but it is equally applicable to the future use of existing works. It can be applied to a complete works, or to a specific assembly, component or system, such as plant or a roofing assembly.

The life cycle cost for a complete building or structure is built up from the sum of the independent parts plus the interaction between them and the consequential costs, if any. Life cycle costing can be carried out at a gross level using industry average figures for that type of construction, or at a specific level on the basis of specific estimates or predictions of component performance and required maintenance activities. Calculations of life cycle cost can be made at various levels depending on which phase of the project process is entailed. Similarly, the degree of detail and information available will play a decisive role. The general principle that determines the level at which calculations of life cycle cost are made is the corresponding level of detail employed to calculate the acquisition costs.

Repercussions or consequences of selecting an alternative must be considered, e.g. a change in the thermal resistance properties of different claddings may result in changes in heating and cooling costs, changing from a paint to a lightweight stain may have a different regime of coating, and different frequencies. In particular, options may have different external or intangible costs, such as access, disruption to building activities, or costs borne by others than the client (e.g. retail tenants to a shopping centre during building maintenance).

3.3.5 Inputs needed for LCC analysis

Attempts to obtain accurate life cycle costs should be consistent with both the ability to predict these costs and the potential for these costs to influence the decision being taken, as well as the size of the investment. Values for the costs can be derived from:

- direct estimation from known costs and components
- historical data from typical applications
- models based on expected performance, averages etc
- best guesses of the future trends in technology, market and application.

For each cost, whether a cost element or a detailed cost category, there must be an associated time or times during the service life when the cost occurs (or recurs) for life cycle costing to be carried out. Time profiles of the costs may only consist of one occurrence but any cost spread over time or one which is repeated will generate a series of cost and time pairs. Costs may be fixed or variable over time. These values are most readily converted into life cycle costing calculations using a computer spreadsheet or purpose written software. Input parameters are:

- Acquisition costs
- Operating costs
- Maintenance costs
- Service life data
- Period of analysis
- External costs and savings data
- Residual/ disposal values/ costs
- Economic parameters
- Discount and inflation rates
- Taxes and regulatory expenses
- Intangibles
- Environmental impacts

3.3.6 Decision variables

The concepts and formulae of LCC calculations are well known and the draft draws on established documents such as ASTM E917 (1994) and ASTM E833 (1997). Some of the general concepts are briefly discussed here but the reader should refer to the documents above or other references on LCC techniques for specific details.

Costs in a life cycle costing analysis can be in real, nominal or discounted terms. A value in real cost is the monetary amount to be paid if the cost occurred at the base date, regardless of which point in time it occurs. The need to refer to a year when using real costs is because of general price inflation and also the fact that future prices may not necessarily change at the same rate for all items being covered by the LCC. The main benefit in using real costs is that it allows for more accurate estimates of costs since current values are used.

Real costs are not generally appropriate for preparing financial budgets where the actual monetary amounts in today's currency are required to ensure actual amounts in future currency are available at the time when they are to be spent. Nominal costs are ascertained from projected economic, technological and efficiency factors. The nominal cost is related to the real cost by the expected general price inflation.

Discounted costs are calculated by taking costs that occur in future years and reducing them by a discount factor derived from the discount rate. Different discount rates apply depending on whether nominal costs or real costs are being discounted. With nominal cost, the discount rate should include an inflation component. If real costs are used, the discount rate should *not* include an inflation component. Different discount rates also apply to different organisations and individuals. The discount rate reflects the preference for money now rather than later. The discount rate is the interest rate that would make it just worthwhile for the decision-maker to spend money in a year's time rather than now. It is also the rate by which future income would need to be discounted each year so that the decision-maker would prefer to have it now rather than in a year's time.

When evaluating life cycle costs, two different terms may be used to enable comparison of different options. Present value is the cost found by discounting future cash flows to the base date. It is used for comparing alternatives with the same period of analysis. Where options have a different period of analysis, the annual equivalent cost is used.

3.3.7 Uncertainty and risk

There are uncertainties inherent in LCC methodology and it is necessary to guard against gross errors in the results. The degree of confidence in the results of LCC analysis depends on the quantity and quality of the data used as input into the analysis. For the acquisition phase, cost figures for various options can usually be estimated with some degree of accuracy since they are derived from current costs. Estimates of operating costs are also reasonably reliable for the new structure but become less certain as the time span increases due to uncertainties in energy costs, labour and regulatory changes (e.g. environmental legislation).

The use of very long periods of analysis should be avoided since the use of times in excess of about 30 years tends to result in costs beyond this becoming negligible in today's dollars in many cases. In order to take account of risk and uncertainties, sensitivity analyses can be undertaken to examine the impact of credible variations in the data used in the LCC calculation. These ranges should be likely, within the limits of what is anticipated, and fit within the client's brief. This will identify which input data have the most impact on the LCC result and how robust the final decision is. Data that should be subjected to sensitivity analysis includes:

- discount rates
- period of analysis
- maintenance regime frequencies
- incomplete or unreliable service life data based on assumptions.

Sensitivity analysis entails iterating the calculation with a range of values for the data variable. The assessment will indicate the vulnerability of the LCC to variations in this respect. If the sensitivity analysis indicates that the alternative variables have no effect on recommendations the decision will be unaffected. If however the recommended option is varied by different rates/lives etc. being applied it may indicate that further analysis is required or that the decision is based upon factors other than LCC.

3.3.8 Reporting

The draft contains some basic guidelines for reporting to ensure that users can clearly understand both the outcomes and the implications of the analysis, including limitations and uncertainties. The report should include:

- purpose and scope - a statement of objective, materials under consideration, assumptions, constraints, and alternatives
- intangibles - impacts likely to affect costs either directly or indirectly but which can not be quantified
- life cycle costing analysis - a presentation of the results, including sensitivity analysis, cost structure and method of accounting for the time value of money
- discussion - a thorough discussion of the interpretation of the results
- conclusions and recommendations

Reference to ISO 15686 Part 3 Auditing is also made at this point.

3.3.9 Bibliography

The draft makes use of a number of current publications on LCC techniques. These include ASTM E917 (1994) and a draft AS/NZS Standard (committee OB/11) on LCC. These are listed in a bibliography.

3.3.10 Annexes

A series of annexes has been included in the draft to show how the LCC calculations are undertaken. The first annex illustrates the use of present value to compare the costs of two cladding options with the same service life and period of analysis. The second annex illustrates the use of the equivalent annual cost method for a case where the service life of two components is different. The third annex illustrates parts of a sensitivity analysis where the effects of varying discount rates and service lives are calculated for a specific example.

4 Conclusions

The draft document provides a framework for further discussion and refinement by the full Working Group and SC14. The committee is actively seeking input from recognised international experts on LCC to help ensure that the draft Standard has a sound technical base before it is circulated widely within SC14, and before it goes for wider comment. A critical input into LCC is data on the service life of materials and building components. This is covered in Parts 1 (via the factorial method) and 2 Service Life Prediction of ISO 15686. The data from Parts 1 and 2 along with the outputs of the LCC are essential to the formation of a sensible maintenance plan for the building. The provision of detailed guidelines on maintenance planning procedures and techniques within ISO 15686 has not currently been addressed and needs further consideration. Environmental considerations also have the potential to significantly impact on LCC evaluations and the development of Part 6 of ISO 1586 will provide additional tools to integrate environmental aspects into LCC exercises.

5 References

American Society for Testing and Materials. (1993) Guide for selecting economic methods for evaluating investments in buildings and building systems. ASTM E1185. Philadelphia.

American Society for Testing and Materials. (1993) Practice for measuring net benefits for investments in buildings and building systems. ASTM E1074. Philadelphia.

American Society for Testing and Materials. (1994) Standard practice for measuring life-cycle costs of buildings and building systems. ASTM E917. Philadelphia.

American Society for Testing and Materials. (1997) Standard terminology of building economics. ASTM E833. Philadelphia.

142 ESTIMATING SERVICE LIVES USING THE FACTOR METHOD FOR USE IN WHOLE LIFE COSTING

K. Bourke,[1] H. Davies[2]

[1]BRE Ltd., Centre for Whole Life Performance, [2]Hywell Davies Consultancy, UK

Abstract

Construction projects have traditionally been costed on the basis of the initial capital cost. The privatisation of many infrastructure providers, the use of private equity to finance construction projects for the public sector and ever-tighter constraints on public spending have generated demands in the UK to predict the overall cost of a project over its whole working life. This concern extends to demands for predictable building service lives and costs of ownership. Until very recently there have been no standard methodologies for approaching this problem. This paper explores the concepts included in whole life costing, in particular that of factored estimates of service life, and reports on the current commercial and practical issues driving adoption of whole life cost procurement of construction works.

Keywords: Factor method, service life estimates, whole life costs, standards, construction procurement, risk.

1 Introduction

Construction projects have traditionally been costed and funded on the basis of the initial capital cost. Privatisation of many infrastructure providers, use of private equity to finance construction projects for the public sector and ever-tighter constraints on public spending have generated demands in the UK and elsewhere to predict the overall cost of a project over its whole working life. This concern extends to demands for predictable costs of ownership. Until very recently there have been no standard methodologies for approaching this problem.

ISO TC59 SC14, Service life planning, is developing an International Standard, ISO 15686, Service Life Planning of Buildings. The first part is entitled "General Principles". It provides a methodology for defining service life requirements during design and a basis for evaluating the ability of the design to meet the design life requirements of the client. In combination with part 5, Maintenance and Life Cycle Costing, (currently under development), this will give

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a methodology for evaluating whole life costs, enabling designers or clients to address whole life cost issues and to estimate costs of ownership of a proposed project at an early stage of design.

Part 2 of the standard provides a generic methodology for predictions of component life based on known performance and rate of deterioration under proposed service conditions; part 3 addresses auditing of service life plans; part 4 covers data formatting and reporting and part 6 addresses sustainability in relation to service life planning.

This paper explores the concepts of whole life costing and estimation of service life, and reports on current commercial and practical issues driving adoption of whole life cost procurement of construction works.

2 Estimating service lives - ISO 15686 factored estimates

The Standard describes two approaches to the prediction of service life. Ideally, service life should be predicted from the known performance and rate of deterioration of the building and its components under the proposed service conditions. A generic methodology for such predictions is given in Part 2 of the Standard. However, the information required to make such a detailed prediction may not be available. In such cases, a method of estimating the service life, taking account of the available data, is available. This method is known as the Factor Method.

2.1 Reasoning behind the factor method of estimating service lives

The Factor Method is based on a concept developed by the Architectural Institute of Japan (AIJ, 1993) and Construction Audit Ltd (CAL, 1992). It is intended as a means of estimating expected service life when more accurate scientific prediction is not possible. It seeks to provide a common framework for developing estimates of service life in the absence of more reliable data. It is therefore an attempt to break out of the cycle of lack of data inhibiting service life prediction and lack of service life prediction suppressing demand for data, first proposed by Heller (Heller 1968).

A discussion paper by BRE (Bourke & Davies, 1997) gives details of the theoretical basis for the system, emphasising that the priority was to produce a simple system which could be readily adopted in advance of substantial extensions to the admittedly limited knowledge base on long-term performance of components. The proposed system has been modified in the light of comments from the international committee steering development of the standard, and comments from both academic and industry reviewers within the UK.

The adoption of the approach was considered to offer a means of forming a considered and objective assessment of alternative options or design solutions, but it was recognised that the performance assessment offered would not be wholly reliable. Since that date further commentary on the method has been offered, and several issues have been raised as potential changes or modifications to the approach. In particular, a paper by Hovde (1998) indicates proposed areas to improve reliability and user-friendliness, and a paper by Hed (1998) gives details of a project where the method was assessed.

Comments have indicated some unease with the factor method as set out in the draft ISO. Concerns include the misapprehension that the method provides a single value answer, rather than a range or distribution; the element of individual subjective judgement in assessing values for factors, and the possibility of providing misleading results. Each of these criticisms has value, and the method needs to be used with due attention to the changing state of the art of service life prediction. The standard demonstrates why urgent production of a simple working methodology, which recognised the limited knowledge base, dictated certain features of the approach. Criticisms have not prevented its fairly rapid adoption within the context of whole life costing of construction. Indeed, whilst the method has limitations, no other more suitable method of estimation has been proposed for use in the absence of accurate data.

2.2 Elements within factored estimates

There are three critical elements within a factored estimate: the required design life, the reference service life and the factors used to amend the reference service. If the estimate exceeds the required design life required then the proposed design meets the performance required.

The three elements in the estimating process are as follows:

2.1.1 Required Design Life

The required design life is the length of time for which the component is expected to remain serviceable, and is derived from the overall design. It is based on the client's requirements.

2.1.2 Reference service life

The reference service life (RSLC) is the period in years that the component or assembly can normally be expected to last under specified service conditions. The Standard gives various sources for the basis of this assessment - but no schedule of reference service lives is provided - it was considered impossible to identify on an international basis what the normal anticipated performance would be. National schedules have been suggested as a possible route forward, and at least one standards committee in the UK has proposed a set of reference service lives for windows and associated sub-components. There are also commercially produced lists of expected service life for some components in particular building types, such as the Component Life Manual (CAL, 1992)

2.1.3 Modifying factors

The modifying factors represent deviations from the specified service conditions used to identify the reference service life - they cover materials, design, sitework, the environment, operating characteristics and maintenance. They are used to modify the reference service life by compound multiplication, as follows:

ESLC = RSLC x A x B x C x D x E x F

Where:
ESLC = Estimated service life of component
RSLC = Reference service life of component
A = Material / Component factor
B = Design factor
C = Workmanship factor
D = Internal Environment factor
E = External Environment factor
F = In-use factor
G = Maintenance factor

Two particular aspects of the factors need to be stressed. First, there is no pre-defined set of factors. The user of the method needs to consider the particular circumstances of the project and decide the most appropriate factors for those circumstances. For example, the factor applied to placement of in-situ concrete in the Arabian desert in summer will be considerably different from that applied to take account of driving rain in Scotland. The method requires intelligent application if the old computing adage is to be avoided: garbage in, garbage out!

The method therefore provides a structured method for identifying and evaluating the factors which will increase or reduce the performance of the component or product under the likely service conditions. It is a method which needs to be applied by suitably experienced designers: it is not appropriate for slavish application of preset figures, but for careful and considered application to well defined problems by suitable professionals. Where there is insufficient data for firm prediction of service life this approach provides a reliable and structured method for estimating the service life. Where clients are insisting on a design with a whole life cost model such an estimate is essential.

3 Whole life costing definitions – how does it compare with life cycle costing?

Once it is established that a design option exceeds the required service life, it must be established whether it is the optimum option or not on the basis of predicted whole life costs. Whole life costing (WLC) provides a "rationale for choice in circumstances where there are alternative means of achieving a given object, and where those alternatives differ not only in their initial costs, but also in subsequent operational costs" (Seeley, 1996). It is not a new approach in industry and has up until recently often been called Life Cycle Costing (LCC). The difference lies in the fact that WLC seeks to consider the whole building life and not just the period of commercial interest.

Essentially WLC is a means of comparing options and their associated cost and income streams over a period of time. An alternative definition on maintenance management (BSI, 1993) stresses that it is "for the purpose of making decisions". The decisions involve considering future events which can be very diverse e.g. how long the building will be needed and what the long term local climate will be like. There is a lot of uncertainty in the results but they do provide a method of choosing between alternatives on the basis of what we know

now, and what we expect the future will bring. Uncertainties are common to many of the factors, and where one approach is more prone to uncertainty than another there is nothing to prevent this uncertainty being expressed in the final results. Estimates should always be subjected to some straightforward sensitivity analysis.

3.1 Who uses whole life costing ?

Whole life cost procurement in construction has been strongly advocated in the UK by long-term building owners and clients in recent years. This includes the 12 major UK construction clients (CRT, 1997), and by the association of most major construction clients in their statement of procurement policies (CCF, 1998). Clients who own and manage buildings long-term want to know their cost of ownership before being committed to a particular building or design alternative. "Avoidance of unpleasant surprises is everything" as the Construction Clients Forum state.

Recently various contractual procurement initiatives have been instituted in the UK, all of which shift long term costs of running built assets from the public to the private sector - for example the Private Finance Initiative (PFI), Public Private Partnerships (PPP), Design, Build, Finance, Operate (DBFO) arrangements etc. These are driving the adoption of WLC by constructors and designers.

A recent survey undertaken by BRE for UK government Department of the Environment Transport and Regions (DETR) indicates that whole life costing is currently used extensively only in PFI projects, and in public procurement projects, and is most frequently undertaken at the early stages of procurement. The major barriers to adoption are cited as the lack of methodology and insufficient data. Other work shows widespread interest in the approach amongst clients and more forward thinking parts of the construction supply chain. The draft ISO standard provides a methodology: the factor method provides a means of estimating service life. Those clients who require greater rigour in their predictions and whole life cost models will need to apply pressure on the supply side of the industry to begin to develop accurate service life data to support whole life costing.

3.2 What is it used for?

Once the life has been estimated, it is possible to assess the various costs that will apply during the service life of the building, including capital or initial cost, maintenance costs and, when appropriate, replacement costs. All the costs associated with the specific option are then added together to give a total cost, with future costs discounted to give a present value. The various options can then be compared on whole life cost grounds. Again, if the process has been completed in a transparent way, it is possible to compare the uncertainties applying to the various options. Part 5 of draft ISO standard aims to address these issues.

3.2.1 Which costs are considered?

Costs to be taken into account include initial capital or procurement costs, opportunity costs and future costs, maintenance or replacement. Only options, which meet the performance requirements for the built asset should be considered: those with lower costs over the period will be preferred.

Initial costs include design, construction and installation, purchase or leasing, fees and charges.

Future costs include all operating costs, such as rent, rates, cleaning, inspection, maintenance, repair, replacements / renewals, energy and utilities use, dismantling, disposal, security and management over the life of the built asset. Loss of revenue may also need to be taken into account – reflecting the non-availability of the revenue-generating building during maintenance work for example.

The timing of future costs must be taken into account in the comparison of options. Future cost flows are discounted by a rate that relates present and future money values – which may include an allowance for inflationary changes. Opportunity costs represent the cost of not having the money available for alternative investments (which would earn money) or the interest payable on loans to finance work.

To convert real costs to discounted (present day) equivalents a discount rate(*f*) is applied as follows:

$$f = \frac{1}{(1 + d)^n}$$

and d = the discount rate per annum
n = the number of years between the base date and the occurrence of the cost

3.3 The essential features of whole life costing

There are several important elements to WLC calculations, and if they are incorrectly assessed the results will be misleading.

3.3.1 Discount rates and inflation

The discount rate is used to represent the present value of a future income stream or cost. It is specific to the client, and includes the degree of risk return required in a commercial context, or the rate of interest payable where loans are required to finance the construction work. If it is set too high, future costs will appear insignificant and will be favoured by the calculation. If it is set too low, capital costs will discouraged, but high operational costs may result. Similarly, sometimes inflation is taken into account in the discount rate, and if rates are substantially different in practice the calculation may lead to inappropriate choices.

3.3.2 Allowances for tax

The tax regime within which the building owner is operating may determine which future costs are allowable for tax – for example capital allowances are currently allowable on new industrial buildings, hotels, industrial and commercial buildings in Enterprise Zones, agricultural buildings and small workshops. Many items of plant and equipment and leased plant are available for allowances, and sometimes associated builders work is also permissible. These allowances also vary depending on the financial situation of the building owner – whether there is a taxable profit against which allowances can be claimed or not.

For major projects specific guidance should be sought on the assumptions made in the calculation, and the applicable rules at the time.

3.3.3 Obsolescence

A building or component may be functional and yet be obsolete, inefficient or unwanted. It is notoriously difficult to assess when or how obsolescence will strike – but changing land values on which the building stands, changing information technology cabling requirements and changing safety requirements have all contributed to render certain buildings obsolete. At present, it is fair to state that rapid technological progress in building services equipment and high fuel costs mean that WLC calculations should assume limited lives for such components.

3.3.4 Building / component life

The period during which the building or component is assessed is an important variable – the building life being considered makes a big difference, because certain major components (such as roof coverings) require replacement at varying intervals. An increase in the required service life of the building of just five years may mean that a major unplanned expenditure is incurred, or conversely if the building is required for a shorter period than planned then high cost, low maintenance options may prove cost-ineffective. Data on the future performance of components are not always readily accessible, and often depend on estimates from accelerated tests. Where a long service life is essential it may be pay to be cautious, and select tried and trusted options. Innovative solutions may however offer good value for money, if the assumptions made are borne out in practice.

Different organisations will have different expectations of building life – and the client may have a limited foreseeable use for the building. Not all clients want or need their buildings to last 60 or 100 years. At the end of a relatively short building life the residual value and/or demolition costs may be significant factors.

3.3.5 Managing uncertainty within the factor method.

Given the accuracy of the information used in this process it is essential to assess the impact of changes in the key assumptions by carrying out a sensitivity analysis. This will establish the degrees of uncertainty in the estimates of service life and, therefore in the whole life costs. The sensitivity analysis can be undertaken to test the impact of different building lives, factors, discount rates and even maintenance costs or replacement costs. Table 1 sets out an example of the sensitivity analysis that might be undertaken to consider the whole life implications of three options available during design or specification. Option 1 considers refurbishment of an existing component, option 2 and 3 considers two quality levels of replacement components.

Table 1: Example of WLC sensitivity analysis

Capital Costs

Option 1 = £203,000
Option 2 = £1,164,900
Option 3 = £1, 262, 100

Discount rate	Service Life	Net Present Values		
		Option 1	Option 2	Option 3
6%	20	1,859,000	1,593,000	1,262,100
	40	2,444,420	1,762,522	2,040,091
8%	20	1,614,000	1,515,000	1,721,100
	40	1,960,950	1,607,896	1,843,068
10%	20	1,423,000	1,451,000	1,638,100
	40	1,632,562	1,505,721	1,709,146

3.4 Improving the balance between capital and operational costs

A key aim of whole life costing is to identify the optimum balance between capital and operational costs. The ratio of operating costs to initial or capital costs varies between building types, and on the selected discount rate and life of the building. However, it is higher than generally considered, and savings in this area are well worth pursuing. Estimated ratios of capital to operating costs can be as high as 1:10 on some UK projects.

4 Conclusions

The prediction of whole life costs of buildings is becoming increasingly important, for economic and environmental reasons. The Factor method has been developed as a tool to support such prediction in cases where there is a lack of adequate or reliable quantitative data. It can assist in the development of whole life cost models, and can help to overcome the lack of data. It must be used with care and understanding of the method and the project under consideration, but is a useful tool to enable constructors to better meet client needs in the early part of the next millennium.

5 References

AIJ (1993) *Principle Guide for Service Life Planning of Buildings.* Architectural Institute of Japan.

Bourke K and Davies H (1997) *Factors Affecting Service Life Predictions of Buildings: a discussion paper.* BRE Laboratory Report 320. CRC Ltd.

BSI (1993) BS3811, *Glossary of Terms Used in Terotechnology*, British Standards Institute.

CAL (1992) *HAPM Component Life Manual*, Construction Audit Ltd., E & FN Spon.

CRT (1997). *Agenda for Change*. Construction Round Table

CCF (1998) *Constructing Improvement*. Construction Clients Forum.

Heller J (1968). *Catch 22*.

Hed G (1998) *Service life Planning in Building Design*. Proceedings of the CIB World Building Congress 1998.

Hovde PJ (1998) *Evaluation of the Factor Method to Estimate the Service Life of Building Components*. Proceedings of the CIB World Building Congress 1998.

Seeley IH (1996) *Building Economics*. Macmillan.

143 REQUIREMENTS FOR DURABILITY AND ON-GOING PERFORMANCE IN CANADA'S OBJECTIVE-BASED CONSTRUCTION CODES

G.A. Chown

National Research Council Canada, Ottawa, ON, Canada

Abstract

The paper describes the nature of objective-based codes, and describes how performance, prescriptive and functional requirements fit into the structure of Canada's new objective-based construction codes.

Canada's national model codes have for some time included a variety of requirements related to durability and on-going performance. The paper differentiates between the concepts of durability, service life and on-going performance, and indicates how these may be addressed in codes.

The paper reviews a number of approaches for incorporating durability and on-going performance requirements into the objective-based code structure and provides examples of the approach that has been accepted to date. The paper looks ahead to the possible development of more specific durability-related provisions.

Keywords: acceptable solutions, building codes, deterioration, durability, failure, functional requirements, objectives, objective-based codes, performance, service life

1 Introduction – background, present, future

The suitability of addressing durability in the National Building Code of Canada (NBCC) and the issues to be considered in drafting durability-related requirements have been discussed (CCBFC 1995a, Chown 1996). The NBCC and Canada's other national model codes are in the process of evolving into objective-based codes. This evolution has raised questions as to how provisions related to durability and on-going performance will be incorporated into the objective-based structures. This paper describes what an objective based code is, outlines options for

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including provisions related to durability and on-going performance, identifies the direction that has been adopted, provides an example of code provisions as they may appear in the objective-based NBCC, and describes opportunities for further development of provisions related to durability and on-going performance.

2 Objective-based codes

Objective-based codes are simply codes whose provisions are related to a hierarchical structure of objectives. The complete code structure consists of four types of elements: objectives, functional requirements, quantitative performance criteria and acceptable solutions (see Figure 1.)

2.1 Root and consequential objectives

All regulations have objectives that they are seeking to achieve. In the case of the NBCC, the two most widely recognized and accepted root objectives relate to the provision of healthy and safe conditions for building users. To address these objectives, one may identify a number of consequential objectives.

In the example illustrated in Figure 1, minimizing the risk of unsafe conditions requires protection from fire and structural failure. To minimize the likelihood of structural failure, adequate initial strength and resistance to deterioration are consequential objectives. To reduce the likelihood of inadequate initial strength, the structure must be designed to resist all dead and live loads that might be imposed.

2.2 Functional requirements and quantitative performance criteria

At the bottom of the hierarchy, the objectives become sufficiently specific that they address particular building elements and provide criteria. These objectives are referred to as functional requirements. In theory, these may be qualitative or quantitative. For example, to resist all of the loads that might be expected to be imposed on a floor, a functional requirement might state:

> Floors in residential spaces shall be designed to resist expected live loads imposed by occupants, furnishings and appliances.

The same functional requirement might be stated so as to include a quantitative limit:

> Floors in residential spaces shall be designed to resist 1.9 kPa.

In the Canadian objective-based construction codes, all functional requirements will be stated in qualitative terms. This is intended to minimize the need to revise these requirements over time as new knowledge is developed that might result in changes to quantitative criteria. Related quantitative performance criteria (QPC) are then included as separate elements in the code structure.

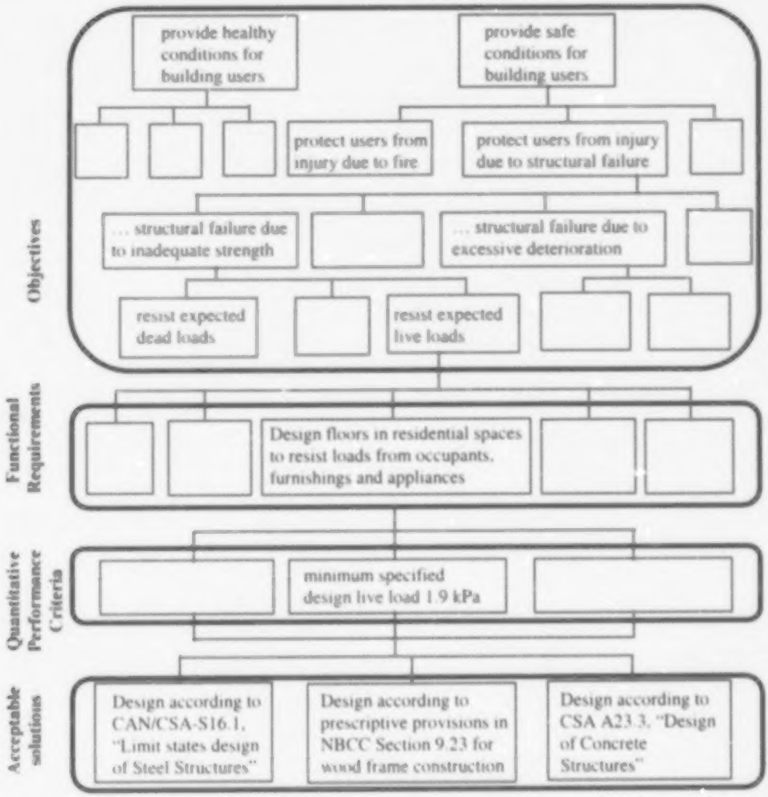
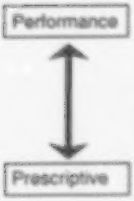

provide healthy conditions for building users
provide safe conditions for building users
protect users from injury due to fire
protect users from injury due to structural failure
… structural failure due to inadequate strength
… structural failure due to excessive deterioration
resist expected dead loads
resist expected live loads
Design floors in residential spaces to resist loads from occupants, furnishings and appliances
minimum specified design live load 1.9 kPa
Design according to CAN/CSA-S16.1, "Limit states design of Steel Structures"
Design according to prescriptive provisions in NBCC Section 9.23 for wood frame construction
Design according to CSA A23.3, "Design of Concrete Structures"
Objectives
Functional Requirements
Quantitative Performance Criteria
Acceptable solutions

Fig. 1: Example of hierarchy of objectives with related functional requirements and acceptable solutions

2.3 Acceptable solutions

To comply with a functional requirement and its related QPC, a variety of solutions may be developed. These are referred to as acceptable solutions and may be performance- or prescriptive-based. Figure 2 lists possible acceptable solutions to a functional requirement concerned with protection from corrosion, illustrating a continuum from performance to prescriptive. Consequently, an objective-based code is not a solely performance-based code or a prescriptive-based code, but allows for the development of numerous solutions, described in performance or prescriptive terms, to meet the functional requirements and achieve the stated objectives.



- treat component as necessary to ensure acceptable performance over the service live of the assembly in which the component is a part.
- treat component to limit rate of material loss to Y % by mass over the service live of the assembly in which the component is a part.
- treat component to limit rate of material loss to X g/kg over 5 years
- apply galvanizing to component to limit rate of material loss to X g/kg over 5 years
- apply 305 g galvanizing per m^2 surface area of component

Fig. 2: Example of continuum of acceptable solutions from performance to prescriptive

Construction that satisfies the provisions of the current codes represent one set of acceptable solutions. For example, the prescriptive requirements currently provided in the NBCC for wood-frame construction of floors would constitute an acceptable solution for fulfilling the functional requirements provided above. Alternative solutions may also be provided by various consensus-based standards.

3 Code provisions for durability and on-going performance

The NBCC has over time included a variety of durability-related requirements and it is agreed that "... requirements for quality and durability that affect health and safety are appropriate" (CCBFC 1995a). It has been agreed that "durability is a factor appropriate for codes provided the concern is related to the objectives of the particular code and any requirements are clear, explicit and enforceable at the time of construction" (CCBFC 1996). In the context of construction codes, it is important to differentiate between durability, service life and on-going performance.

3.1 Durability

For the purpose of this paper, durability of a building material, component, assembly or system is defined as that element's

> ability, at a single point in time and measured in terms of time, to resist or accommodate agents and mechanisms of deterioration in the service environment, and thereby perform the required functions.

Durability depends on initial properties and rate of deterioration in a particular service environment; that is, on initial performance and the rate of deterioration of performance in the service environment. As durability reflects a snapshot in time, it can be considered independent of maintenance. Carrying out maintenance will alter the properties of the building element or its service environment; consequently, durability after maintenance will be different from that before maintenance. (See Figure 3.)

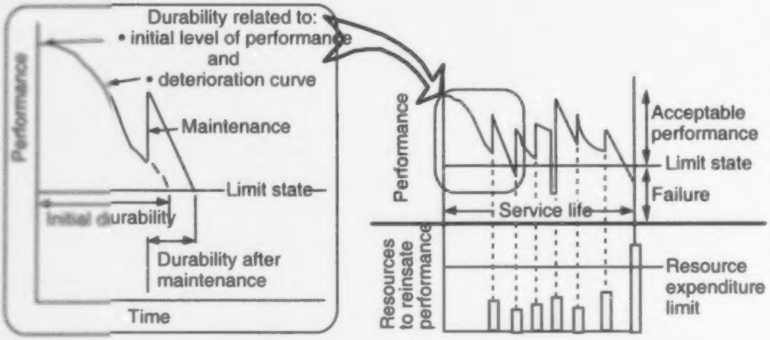

Durability related to:
• initial level of performance
and
• deterioration curve
Maintenance
Limit state
Initial durability
Durability after maintenance
Performance
Time
Performance
Acceptable performance
Limit state
Failure
Service life
Resources to reinsate performance
Resource expenditure limit

Fig. 3: Durability, deterioration and service life (Chown and Oleszkiewicz 1997)

3.2 Service life

Durability and maintenance together determine service life.

> Service life is the period after installation during which all properties of a material, component, assembly or system essential to its required functions meet or exceed minimum acceptable levels without major costs or disruption for major maintenance, repair or replacement.

That is, service life depends on maintenance and the resources necessary to carry out that maintenance. When cost for maintenance or repair exceeds an acceptable level or when replacement is required, service life has ended.

In a building clad with face-sealed precast concrete panels, deterioration of sealant between the panels will allow water entry. Where this indicates performance below the limit state, the cladding system, in effect, fails. Although the sealant has reached the end of its service life, likely the whole cladding system has not. If the sealant can be replaced for reasonable cost, the assembly will continue to function.

Obsolescence, or physical damage or deterioration of the cladding components themselves, may require, to re-establish performance, expenditure of resources that exceeds a certain limit. At this point, the end of the service life of the cladding has been reached. If resources are expended to repair or replace the cladding components, the cladding system would acquire a new service life.

3.3 Addressing durability in building codes

Durability, because it refers to the characteristics of an element at a particular point in time and the expected deterioration curve after that time, may be addressed relatively easily in building codes, such as the NBCC, that only comes into force up until and at the time of hand-over of the building. It may, for example, be expressed prescriptively in terms of material properties required when the element is installed in a particular assembly or exposed to a particular climate.

3.4 Addressing service life and on-going performance in codes

Service life, as it depends on maintenance, is difficult to address in a building code. It may be addressed in regulations that consider on-going operation of a building and its elements. The National Fire Code of Canada (NFCC) (CCBFC 1995b) is an example of a code that addresses on-going performance with respect to fire safety.

In the code context, the concern is not with service life per se but rather with on-going performance. Service life is related to the resources required for maintenance, but resources are not a primary concern of most codes. The NFCC is not concerned with whether on-going performance is provided by a single durable element with a long life or by an element that is maintained, repaired or replaced regularly.

Regardless of the above, there are instances where the term service life appears in the NBCC. The NBCC does not, however, identify accepted means of establishing service life, and design service life need not recorded in design documents.

4 Durability and on-going performance in an objective structure

Because the development of the objective hierarchy for Canada's national model codes is in progress, it is not possible to describe how any one requirement or a set of requirements will be addressed in the final objective-based documents. A number of options for fitting objectives and provisions related to durability and on-going performance into the structure have been discussed.

4.1 Durability and on-going performance as root objectives

Including durability and on-going performance as root objectives, along with health and safety for example, would suggest that durability and on-going performance are objectives in of themselves and are unrelated to the other identified root objectives. This reflects some public expectations but, as noted, is inconsistent with the decisions related to the strategic development of the national model codes (CCBFC 1996).

4.2 Durability and on-going performance as third dimension to the objective structure

Looking upward through the objective hierarchy from some lower level in the structure, development of the hierarchy is based on answering the question "why?" Durability addresses the question of "how long?" An argument was made that durability can be related to each and every root and consequential objective in the hierarchy. To reflect this, durability could be addressed in a structure that is separate from but parallel to the basic objective tree, or as a parameter that may be attached to every node in the hierarchy. This is theoretically valid but implies an unnecessary degree of complexity. In reality, the issues of durability and on-going performance need only be addressed explicitly at certain levels in the hierarchy.

4.3 Durability and on-going performance within the objective structure

Preliminary efforts to link root objectives with code requirements indicate that the NBCC and NFCC address durability and on-going performance at various levels. In some cases, the existing code provisions directly address the subject. In other instances, the relationship between the provision and durability or on-going performance only becomes evident higher in the objective tree.

The following present small slices of the objective-based hierarchies of the NBCC and NFCC to illustrate how durability and on-going performance might be included in those hierarchies. References to durability, deterioration, service life and on-going performance are in bold type to highlight the various levels at which the issue may be addressed. It is important to note that the current codes do not necessarily provide QPC or acceptable solutions in all cases. This issue is discussed further under Opportunities and Challenges below.

CURRENT CODE PROVISION

NBCC 6.2.9.2. Insulation and Coverings

1) Insulation and coverings on pipes shall be composed of material which will withstand deterioration from softening, melting, mildew and mould at the operating temperature of the system.

OBJECTIVE PROGRESSION 1

- provide HEALTHY CONDITIONS for building users
  - protect users from inadequate air quality
  - protect users from inadequate thermal conditions
    - ...due to **premature failure** of the heating or cooling system

Functional requirement

- **6.2.9.2.(1)** Insulation and coverings on heating and cooling pipes shall be **resistant to thermal and biological deterioration**.

Quantitative performance criteria

- (undefined)

Acceptable solution

- (undefined)

OBJECTIVE PROGRESSION 2

- provide SAFE CONDITIONS for building users
  - protect users from injury due to fire
  - protect users from injury due to structural failure
  - reduce probability of user injury due to burning
    - ...due to **premature failure** of protective material over exposed hot elements

Functional requirement

- **6.2.9.2.(1)** Insulation and coverings on heating and cooling pipes shall be **resistant to thermal and biological deterioration**.

Quantitative performance criteria

- (undefined)

Acceptable solution

- (undefined)

CURRENT CODE PROVISION

NBCC 9.20.9.5. Ties for Masonry Veneer

1) Masonry veneer 75 mm or more in thickness and resting on a bearing support shall be tied to masonry back-up or to wood framing members with straps that are a) corrosion-resistant, ….

Objectives
- provide SAFE CONDITIONS for building users
  - protect users from injury due to fire
  - protect users from injury due to structural failure
    - ...structural failure resulting from inadequate strength
    - ...structural failure resulting from excessive rate of deterioration
      - ... **excessive rate of deterioration** due to corrosion
        - control temperatures in the assembly
        - control moisture levels in the assembly
        - provide protection of corrosion-prone materials

Functional Requirement
- **9.20.9.5.(1)(a)** Masonry ties shall be **corrosion resistant**.

Quantitative performance criteria
- (undefined)

Acceptable solution (or part thereof)
- 9.20.16.1. Corrosion Resistance of Connectors, Table 9.20.16.1. provides **minimum prescriptive galvanizing criteria**.

CURRENT CODE PROVISION

NFCC 2.3.2.2. Flame Retardant Treatment

1) Flame retardant treatments shall be renewed as often as required to ensure that the material will pass the match flame test in NFPA 701, "Fire Tests for Flame-Resistant Textiles and Films."

Objectives
- provide SAFE CONDITIONS for building users
  - protect users from injury due to structural failure
  - protect users from injury due to fire
    - ...due to combustion of fabrics
      - ... due to **deterioration** of flame retardants

Functional Requirement
- **2.3.2.2.(1)** Flame retardant treatment shall be renewed to ensure **on-going performance**.

Quantitative performance criteria
- per NFPA 701

Acceptable solution
- **2.3.2.2.(1)** Testing in accordance with NFPA 701.

5 Presentation of information in an objective-based code

Organizing objective-based code material to reflect its logical structure is relatively simple in electronic form but extremely cumbersome in hardcopy. Consequently, the codes will be organized with respect to the subjects addressed, similar to the existing documents.

The following provides one possible presentation of an excerpt from the objective-based NBCC. This excerpt provides information related to controlling heat loss, including potential consequences for deterioration. Information is provided on the application of the provisions, the objectives and the functional requirements. QPC and accepted solutions are not included in this excerpt. Accepted solutions would include sources of climate data, accepted calculation procedures, prescriptive installation provisions, and referenced material and installation standards.

5.1. Heat transfer control

5.1.1. Exposure to intended temperature differential

Application

This Subsection applies to building components or assemblies that will be subjected to an intended temperature differential, except where it can be shown that the effects of the temperature differential will not adversely affect achieving the Objectives of this Subsection.

Objectives

The Objectives of this Subsection are

1) Health
   a) To reduce the likelihood of unacceptable interior air or surface temperature fluctuations due to inadequate control of heat transfer through *environmental separators*
   b) To reduce the likelihood of harmful indoor air pollution from biological growth or from materials becoming unstable due to wetting from condensation of moisture from interior spaces
   c) To reduce the likelihood of premature failure of environmental separators caused by deterioration due condensation of moisture from interior spaces.
   d) To reduce the likelihood that the means of heat transfer control presents a health hazard
2) Safety
   a) To reduce the likelihood of injury from the loss of integrity of building components due to deterioration caused by condensation of moisture from interior spaces
   b) To reduce the likelihood that the means of heat transfer control presents a safety hazard

Requirements

A.5.1.1-R-1

Building components or assemblies identified in Application shall be designed and built to provide sufficient resistance to heat transfer,

a) in conjunction with systems installed for space conditioning, to meet the interior design thermal conditions for the intended occupancy (Objective 1a)

c) to reduce the likelihood of surface condensation on the warm side of the component or assembly to a degree that will not initiate deterioration. (Objectives 1b, 2a)

b) in conjunction with other materials or components in the assembly, to reduce the likelihood of condensation within the component or assembly to a degree that will not initiate deterioration. (Objectives 1b, 2a)

A.5.2.1-R-2

Building materials, components, assemblies or systems installed to control air transfer shall be (Objectives 1a, 1b, 1c, 1d, 2a, 2b)

a) compatible with adjoining materials,

b) resistant to any mechanisms of deterioration which would be reasonably expected given the nature, function and exposure of the building element.

A.5.2.1-R-3

Building materials, components, assemblies or systems installed to control air transfer shall not substantially contribute to the pollution of indoor air (Objective 1d)

A.5.2.1-R-4

Building materials, components, assemblies or systems installed to control air transfer shall not create a safety hazard (Objective 2b)

6 Opportunities and challenges

Because an objective-based code permits a large number of acceptable solutions to meet its objectives, there are numerous means of providing for durability and on-going performance that are not currently specified in the NBCC or NFCC. Furthermore, as noted above, there are instances where the current documents do not provide any QPC or acceptable solutions for some functional requirements. The development of the objective-based codes opens the doors to the development of these criteria and alternative solutions.

Where prescriptive acceptable solutions are included in the current codes, the degrees of durability and on-going performance provided by these provisions are largely unknown. The first challenge in these instances is to determine what are the currently acceptable levels of durability and on-going performance; that is, what are the QPC. Once these are defined, there is a basis for determining whether proposed acceptable solutions will fulfill the functional requirements and the related objective(s). Whether acceptable solutions are incorporated directly into the codes or are included by reference, it is vital that they address all the necessary requirements and objectives.

The transition to objective-based codes is providing an opportunity to review all of the current code provisions and to identify areas where rationalization and further information are needed to assist in their on-going development.

7 References

Canadian Commission on Building and Fire Codes (CCBFC). (1995) *National Building Code of Canada 1995*, National Research Council Canada, Ottawa.

Canadian Commission on Building and Fire Codes (CCBFC), (1995*) National Fire Code of Canada*, National Research Council Canada, Ottawa.

Canadian Commission on Building and Fire Codes (CCBFC), (1996) *Possible Measures to Implement the Strategic Plan of the Canadian Commission on Building and Fire Codes*, National Research Council Canada, Ottawa.

Canadian General Standards Board (CGSB), (1980) CGSB 37-GP-56M, Membrane, Modified, Bituminous, *Prefabricated and Reinforced for Roofing*, Ottawa.

Chown, G.A. (1996) Envelope Durability Requirements in the National Building Code of Canada, Durability of Building Materials and Components 7, *Proceedings of the Seventh International Conference on Durability of Building Materials and Components*, Stockholm.

Chown, G.A. and Oleszkiewicz, I. (1997) The National Building Code: Durability Requirements and their Incorporation into an Objective-Based Structure, Durability of Buildings – Design, Maintenance, Codes and Practices, *Proceedings of the Seventh Conference on Building Science and Technology*, Toronto.

144 INTERNATIONAL STANDARDS FOR SERVICE LIFE PLANNING OF BUILDINGS

G.J. Frohnsdorff,[1] C.H. Sjöström,[2] and G. Soronis[3]

[1]National Institute of Standards and Technology, Gaithersburg, MD, USA; [2]Royal Institute of Technology, Gävle and [3]Swedish Building Standards Institution, Stockholm, Sweden

Abstract

ISO Technical Committee TC 59, Building Construction, is an international standards committee dealing with general rules for performance requirements for buildings. SC14, Design Life, is the subcommittee of TC59 that deals with standardisation of the principles and processes that apply when planning the design life of construction works. The standard being drafted by SC14 will serve designers, owners, potential buyers/investors, educators, students, and others. It points out that, while it is difficult to make precise estimates of service lives, the discipline of making estimates based on the principles presented can help avoid unexpected failures. This paper reviews the current work within SC14 and draws attention to issues and research needs yet to be addressed by the working groups drafting the parts of the design life standard.

Keywords: Buildings, building materials, constructed facilities, durability, service life, standardisation, standards, sustainable building, life cycle cost, performance concept.

1 Introduction

Durability and service life of building materials and products is an area of research and development (R&D) that has gained increasing attention over the last decade. There are several reasons for this including the continuing concern for quality and safety in buildings and a growth in concern for resource conservation in the building and construction sector with attention to the issues of life-cycle costing (LCC), life-cycle analysis (LCA), and

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optimisation of maintenance programmes. From a research point of view, service life prediction is now a recognised scientific activity with fairly uniform methodologies available to help guide the R&D work. Service life prediction is also recognised as being important for engineering applications.

2 International collaboration in service life research in CIB and RILEM

Durability and service life prediction of building materials and products is a multidisciplinary research field. Based on materials science, it involves building technologies and building physics, and also meteorology and environmental research. Modern information technologies such as geographical information systems (GIS) are essential to the modelling of performance of building materials and components over time, characterisation of degradation environments, and user-friendly presentation of service life data.

At the suggestion of two of us (CS and GJF), a joint CIB/RILEM research cooperation on service life prediction was started in 1982; (CIB is the International Council for Building Research Studies and Documentation, and RILEM is the International Union for Research and Testing Laboratories for Materials and Structures). It was aimed at establishing generic methodologies for assessing the service life of building products and buildings, thereby unifying the field from a methodological point of view and creating a basis for quality assurance as it relates to service life in constructed facilities.

From the beginning, the intention was to establish a basis for standardisation in the area. In the first period, 1982 – 86, the work focused on describing the state-of-the-art and establishing a generic methodology for service life prediction. In 1989, the generic methodology was accepted as a RILEM Technical Recommendation (Masters and Brandt 1989) and it is now under standardisation by ISO (ISO /TC59/SC14, Part 2, 1998). In the next period, 1987 – 89, the CIB/RILEM joint work focused on methodologies for the feedback of durability data from practice by inspection of buildings (Sjöström 1990), (Brandt 1990). In the 1990 - 96 period, the work concentrated on further development of the methodologies (Sjöström 1994). The results have been reported in a number of reports and papers and it is intended to compile these publications into a handbook (Jernberg et al. 1993). The present work programme, 1997 – 99, continues the detailing of the methodologies with a focus on information technologies (Jernberg et al. 1993).

3 History of ISO TC59/SC14, design life

The standardisation activity started in 1993 when, at a meeting of ISO TC59 in Atlanta, the working group, ISO TC59/SC3/WG9, Design Life of Buildings, was established. There was a significant European initiative behind this action as a result of the setting up of the Eurocare umbrella project under the European research programme, EUREKA. The strategic goal of Eurocare is to increase the service life of the built environment and decrease yearly life-cycle costs for conservation, restoration and maintenance. Eurocare was among the first to adopt the service life prediction methodology established by CIB and RILEM. In 1991, the three organisations – Eurocare, CIB and RILEM -- took the initiative of discussing the methodology and its standardisation with the Commission of the European Communities and the European Standards Organization (CEN). The methodology was accepted as a guiding concept regarding durability of building products that should be of help in implementing the European Community's Construction Products Directive (CPD). ISO TC59/SC3/WG9 was the result of a CEN/BTS-1 request to ISO and a subsequent proposal from ASTM (the American Society for Testing and Materials) through ANSI (the American National Standards Institute); it benefitted from the Vienna Agreement on co-operation between CEN and ISO. Recently, in 1997, in recognition of its high level of productive activity, SC3/WG9 was elevated to subcommittee status as ISO TC59/SC14, Design Life. Apart from the service life prediction methodology document (i.e., the RILEM Technical Recommendation), the basic documents available to assist the early standardization work were some national standards and codes of practice -- specifically, the British Guide to Durability of Buildings and Building Elements, Products and Components (1992); the Japanese Principal Guide for Service Life Planning of Buildings (1993); and the Canadian Guideline on Durability in Buildings (1994).

4 Status of the work of ISO TC59/SC14, design life

The first part of the proposed ISO standard (ISO/TC59/SC14 Part 1, 1998) has been described by Browne and Soronis (1996). It gives guidance for building designers on the service life planning of buildings. It is expected to be approved as a DIS (Draft International Standard) early in 1999. The voting on the document's DIS status will be completed in December 1998. Part 2 of the standard: Service Life Prediction Principles (ISO/TC59/SC14 Part 2, 1998) is also expected to become an ISO/DIS early in 1999. Subsequent parts of the standard will focus on Auditing, Maintenance, Life Cycle Costing, and Data Formats. In addition, a work item for guidelines for the inclusion in the standard of matters relating to LCA has been initiated. A vision of the standards that need to be developed over the long term has been provided by Frohnsdorff and Martin (1996).

The standardisation activities in ISO TC59/SC14 are strongly supported by the work of the joint CIB W80/RILEM 175-SLM Committee on Service Life Prediction, with SC14 being seen as the primary customer for the products of the committee.

5 Prenormative research needs

The following R&D areas are crucial for the future implementation of the ISO standard:

- *Performance analysis.* A systematic, internationally-accepted methodology for preparing lists of performance requirements for important types of construction products is urgently needed.
- *Characterisation, mapping, classification, and development of databases of environmental degradation factors.* Reliable and comprehensive environmental data suitable for use in service life predictions are needed for all regions of the world. The

environmental degradation factors should be classified, mapped and presented by GIS in a user-friendly form; (ISO 9223:1992 provides a standard classification system for corrosivity of environments for some metals that might be used as a model). To meet the need for data, a network of standardised, well-instrumented field test sites in various climatic and environmental zones should be established. The sites in such a network should be linked together to provide reliable and comprehensive environmental data. A basis for a network exists through the United Nations ECE International Co-operative Programme for Materials; the Programme has operated a well-monitored network of approximately 40 test sites throughout Europe and North America for the last ten years. The present monitoring programme is only scheduled to run until 2001. A number of co-operative initiatives are based on, and connect to, the UN/ECE Programme and focus on areas such as corrosion mapping, air quality, and cost-benefit analysis.

- *Identification and modelling of degradation mechanisms and dose/response functions.* The lack of knowledge of exposure/degradation environments and of dose/response functions relating the response of materials and components to the environment are major barriers to reliable service life predictions. In the environmental research area, knowledge and data that have been generated over the years have been used in cost-benefit analyses of the degradation of the built environment and as input to policy decisions on abatement strategies. However, much more data will be needed before service life predictions can become a routine aid to designers and others. The considerable body of data on exposure conditions that now exists includes European data on the macro and meso levels, and dose/response functions have been established for a number of important building materials (Haagenrud and Henriksen 1996). A major task will be the transition from knowledge based on dose/response functions to predictions of service life and performance over time for real buildings and other constructions. Building inventories and testing under in-use conditions are needed in this process to refine and further develop the knowledge base. Dose/response functions should be validated in the micro environment of buildings.
- *Development of long-term and short-term test facilities and programmes.* Special laboratory facilities are needed for studies of factors affecting the mechanisms and rates of degradation of materials and components, and development of relevant test methods where needed. These include environmental chambers of various types in which materials can be exposed to various combinations of closely-controlled conditions in applying the service life prediction methodology. It should be noted that, contrary to the widely-accepted notion, appropriately-selected reproducible laboratory conditions, not irreproducible outdoor exposures, must ultimately become the standard of reference for service life predictions. Dose-response functions based on laboratory data may then be combined with knowledge of typical environmental conditions at a site to make service life predictions for materials that are to be used at that site. Protocols for relating the results of artificial aging to those of natural weathering must be developed.
- *Life-Cycle Cost and Life-Cycle Assessment models.* Research is needed on life-cycle cost models to connect service life of materials and components to the economic performance of the building during its whole working life. It is internationally accepted that the rapidly growing interest in life-cycle assessment of building products will require reliable service life data if LCA is to be most useful (Soronis and Makenya 1998).

- *Information Technologies (IT).* It is essential that IT systems be developed to facilitate incorporation of the service life prediction methodology into the building design process. The systems must be user-friendly computer-integrated knowledge systems consisting of databases, simulation models, and knowledge-based expert systems established according to standard guidelines; GIS-based information systems would be included (Haagenrud and Henriksen 1996). It is to be expected that most of the laboratory data for materials and components in the knowledge systems, and much of the material degradation data from test sites and actual buildings, will eventually be provided by the manufacturers of the materials and components.

6 Some concluding remarks concerning implementation of service life planning standards

The development and implementation of service life planning standards are complex tasks that should be considered together. The following remarks relate to implementation and further development of the ISO service life planning standard:

- There is sufficient knowledge and data to show that standardisation regarding prediction of service life in the built environment is a realistic, though challenging, goal; however, much more data will be needed to obtain the full benefits from the standard.
- In some cases, modelling and mapping of environments and the response of materials (in a building assembly) to exposure in an exterior environment are possible using available tools and data.
- International efforts are needed to provide the data and knowledge that will benefit all regions of the world by increasing the service life of the built environment and decreasing life-cycle costs.
- Service life research is interdisciplinary and some of the necessary knowledge and tools, such as materials science, environmental characterization, and information technologies, lie outside the traditional building sciences; R&D programmes must be organised accordingly.
- Most of the research needs relaying to service life prediction and planning are of a prenormative character; encouragement of prenaormative research is vital.
- It is necessary to ensure that the ISO standard on design life and service life prediction is consistent with the life-cycle analysis standards in the ISO 1440 series.

7 References

Masters, L.W. and Brandt, E. (editors) (1989) Systematic methodology for service life prediction of building materials and components, *Materials and Structures*, 1989, 22, 385 – 392

ISO TC59/SC14 (1998), Buildings - Service Life Planning - Part 2: Service Life Prediction Principles, *Second Committee draft*, 1998-08-31

Sjöström, Ch. (editor) (1990) Feedback from practice of durability data: Inspections of buildings. *CIB report, publication 127.*

Brandt, E. (editor) (1990) Feedback from practice of durability data: Appendix. Examples of field investigations. *CIB report, publication 128*

Sjöström, Ch. (1994) CIB W80/RILEM 140-TSL on Service life prediction. – Methodologies and research needs. Proceedings of Vision EUREKA Conference *New technology for Management and Maintenance of Old and New Built Environment*, Lillehammer, Norway, June 1994.

Jernberg, P., Sjöström, Ch. And Lacasse, M. A. (1997), TC 140-TSL: Prediction of Service Life of Building Materials and Components (Joint with CIB W80). State-of the-art-report, *Materials and Structures*, supplement March 1997, pp 22 - 25

ISO TC59/SC14 (1998), Buildings - Service Life Planning - Part 1: General Principles, ISO/DIS 15686-1

Browne, R. and Soronis, G. (1996) A proposed ISO-standard for service life of buildings: Part 1 General Principles. *Durability of Building Materials and Components 7*, (ed. Ch. Sjöström), E & FN Spon, London

Soronis, G., Makenya, A. (1998) Towards a new integrated ISO-standard for sustainable building, *Proceedings of CIB World Building Congress 1998*, Vol. 1, Gävle Sweden

Frohnsdorff, G. and Martin, J. (1996) Towards prediction of building service life - The standards imperative. *Durability of Building Materials and Components 7*, (ed. Ch. Sjöström), E & FN Spon, London.

Haagenrud, S.E. and Henriksen, J.F. (1996). Degradation of built environment – Review of cost assessment and dose response functions, *Durability of Building Materials and Components 7*, (ed. Ch. Sjöström), E & FN Spon, London.

International Organisation for Standardisation (1992) Corrosion of metals and alloys – Corrosivity of atmospheres - Classification. ISO 9223:1992.

Haagenrud, S.E., Henriksen, J.F. and Skancke, T. (1996) Modelling and mapping of degradation of built environment from available data and GIS based information tools. *Durability of Building Materials and Components 7*, (ed. Ch. Sjöström), E & FN Spon, London.

British Standards Institution BS 7543:1992. Guide to Durability of Building Elements, *Products and Components.*

Architectural Institute of Japan (1993). The English Edition of Principal Guide for Service Life Planning of Buildings.

Canadian Standards Association (1994). Guidelines on Durability in Buildings, S-478-1994.

145 SERVICE LIFE PLANNING OF BUILDING COMPONENTS

G. Hed

KTH (Royal Institute of Technology), Gävle, Sweden

Abstract

Service life planning of building components was studied for a whole building in an R&D project, carried out at the Centre of Built Environment, Gävle, Sweden. The study was connected to a demonstration construction scheme and was integrated in the design and construction of a building. The study was performed in accordance with the draft standard ISO/DIS 15686.1 Buildings: Service life planning, and one aim was to test, evaluate and give input to further development of the standard. To make an estimation of a component's service life, the requirements, degradation environment and performance have to be considered. This paper will discuss this approach and discuss how available service life data can be collected and evaluated. Examples of service-life predictions are also shown.

Keywords: Service life planning, service life predictions, degradation environment, degradation agents, degradation mechanisms.

1 Introduction

This paper describes the use of a methodology for service life planning of building components and materials. The work was carried out on an actual building project, a four-storey apartment house in Gävle, Sweden. The service life planning is carried out in accordance with the draft standard ISO/DIS 15686.1 Buildings: Service Life Planning, part 1: General Principles (ISO 1998). The standard defines how to find out whether an estimated service life (ESLC) meets or exceeds a desired service life (DLC, design life of component). This paper is a continuation of (Hed 1998) where the early stage of the service life planning is described. Background and preparatory

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work of the ISO standard are presented by (Caluwerts 1996) and (Soronis 1998).

To perform the service life planning the following aspects of a component should be considered:

- requirements of the component and its design life
- type and intensity of degradation agents
- performance – the reaction of materials to degradation agents.

An ideal situation is that data are available on all of the three aspects and that the data are accurate and well defined. But that is not always the situation today; very few service-life data are available. Data that are at hand will be used in the estimations although they may not be as accurate as desired. This is accepted in the project and this shortage will be used to point out further research. Results of estimations are shown as examples.

2 Method of service life design

2.1 Performance requirements

It is essential to describe the function and the requirements of every component since the service life of a component is reached when these requirements are not met. The same approach applies for all parts of the building such as structure, building envelope, services, complements etc. When establishing the requirements of a component it is essential to define the level of the component on which the function of the component should be described (see Table 1).

Table 1: Level of component

Level	*Example*
Building Assembly	Facade including rendering, windows, ventilation openings, fixings, flashing etc.
Component	Facade rendering
Material	Top coating
Molecule	Chemical composition of top coating

The next step is to describe the performance requirements. The requirements can be of the type either/or such as a non-acceptance of a water leakage or a degradation such as a maximum reduction of a paint-thickness due to weathering agents (see Table 4). At this stage the design life of the components (DLC) are set. As governing user requirements, the six essential requirements in the European Union Construction Product Directive, CPD are employed (Caluwerts1996).

2.2 Degradation agents – degradation environment

Degradation agent is defined in ISO/DIS 15686-1 as: "Whatever acts on a building or its parts to adversely affect its performance, e.g., person, water, load, heat." This means that wear for a flooring material and use such as opening and shutting of a window are also degrading agents. Degradation environment is the combined action of different agents that act on a material. The degradation agents originating from the atmosphere can be classified in four levels, see table 2 (Haagenrud 1997).

Table 2: Levels of degradation agent originating from the atmosphere

Level	*Example*
Macro	Global: Sweden, Northern Europe
Meso	City, community: Gävle
Local	Location in city: Kvarteret Diligensen (block in actual building project).
Micro	Actual location of a component on the building.

At the meso-level, climate data can be obtained: wind speed, predominant wind-direction, driving rain, temperature characteristics, relative humidity etc. In Table 3, air pollution characteristics are described for the site of the building on a meso/local level. Subsequently these data will be interpreted to a micro-level, which is dependent on the component location on the building and also the building material itself.

Table 3: Air pollution characteristics for Gävle, Sweden, (Meso)

UN ECE ICP *Agent*			ISO 9223-9226 Category		
TOW	3300	hours/year	T_4	2500-5500	hours/year
SO_2	5-10	µg/m^3	P_0	≤10	µg/m^3
NO_x	30-65	µg/m^3			
Cl^-			S_0	≤3	mg/m^3/day

2.3 Performance over time data

To estimate the service life at last, it is necessary to describe how materials or components react to different agents. Different ways of attaining this are discussed in this section.

- Degradation mechanisms
- Dose response functions
- Evaluation of test results
- Systematic observations on the built environment
- Observations on the built environment

The service life can be obtained by performing calculations based the *degradation mechanism* of a material, where the mechanism can be described on a molecule or a micro level. A typical example of this is when colour changes are explained on an atomic level. Another example is the theory of frost heave to explain degradation of rendering. In this case the requirements either have to be formulated in terms of the theory or the results have to be interpreted into other (visible) terms.

Another approach uses *dose-response* functions where (typically) loss of material is measured and the environment monitored, often for a number of years at different exposure sites. The observations are subsequently correlated into formulas, which can be used to calculate the loss of material for other degrading environments. A dose response function can be of the type $M=A_1*A_2*b^t$, where M is loss of material, A_1, A_2 and b are dependent on material properties or the environment and t is time. If $M= M_0$, where M_0 is a specific value, the time corresponding to this loss of material is the service life.

Conclusions might also be drawn from *tests carried out by component manufacturers*. Examples are flooring abrasion tests, mechanical function tests of doors, tests of frost resistance, acid resistance, fungi resistance etc. Performed *systematic inspections* on the built environment can also be used for estimations, (Tolstoy 1989) and (Sjöström 1990). Information is also available from some insurance companies; one example might be the HAPM Component Life Manual (HAPM 1992).

A considerable amount of service life data are available where the requirements, the degradation environment or the performance are poorly described. These data are often based on observations on the built environment, but are not systematic.

To obtain performance over time data seems to be the most difficult part of the process. It is easier to formulate requirements and describe degradation agents than to calculate the effect or formulate a theory, which also has to be tested for validation.

A general methodology for service life predictions is described in ISO/DIS 15686-2 (ISO 1999). A test program carried out according to that procedure will produce a quality assured predicted service life based on specific conditions. To date few values are available, which are gained with the method in its entirety. However, the knowledge will increase as the methodology is further used in R&D projects and programmes.

2.4 Factor method

A predicted service life as obtained according to the methodology [ISO 1999] or from other sources of information should be used in the design as a reference value in order to estimate the service life of a component. If the design conditions are the same or substantially similar to the reference conditions, the reference value can be used directly as estimation. But what to do if the conditions deviate from the reference conditions? ISO/DIS 15686-1 suggest a factor method (discussion see Hovde 1998). The service life of a component is estimated by using the formula

$$\mathrm{ESLC} = \mathrm{RSLC} \times A \times B \times C \times D \times E \times F \times G \qquad (1)$$

ESLS = Estimated service life of component
RSLC = Reference service life of a component.

The modifying Factors A to G reflects the deviations from the conditions from which the RSLC derives.

- A Quality of components
- B Design level
- C Work execution level
- D Indoor environment
- E Outdoor environment
- F In-use conditions
- G Maintenance level.

3 Results

Worked examples of service life estimations are shown in Table 4. Of significance is the facade rendering seems to have a longer service life than the flashing. This leads to the conclusion that either a maintenance program or a material change needs to be done for the flashing.

Table 4: Examples of results of service life estimations

Component	*Performance*	*Performance criterion*	*DLC*	*Degrading agent*	*Estimation of service life*	*Comment*
Exterior door, SWEDOOR						
Hinges, lock	Mechanical function. Open, shut and locking of door.	Non-acceptance of malfunction	60	Open shut 10 times a day (200 000 times)	Life expectancy 60 years with maintenance	Tests performed by manufacturer Feedback from experience
Draught stripping	Air-tightness	Non-acceptance of malfunction	30	Weathering, use	Maintenance interval 30 years	See above
Paint	Protection of substrate	Protection ability (reduction of thickness, cracking adhesion etc.)	10	Weathering	Maintenance interval 10 years	See above
Paint	Aesthetic, colour changes	According to standards (reference pictures)	10	Weathering	Maintenance interval 10 years	See above
Flashing, (Figure 1,2) coil coated steel	Weather protection, covering of joints	Non-acceptance of malfunction	60	Weathering		
Front coating (Plastisol)	Protection of substrate	Protection ability (blistering, cracking adhesion loss)	30	Weathering	10 years, maintenance interval	(Sjöström 1990)
Front coating	Aesthetic, colour changes	According to standards (reference pictures)	30	Weathering	10 years, maintenance interval	(Sjöström 1990)
Zink	Loss of material	20 µm	30	Weathering (air pollution)	Service life 25 years (no coating)	Dose/response calculation (Haagenrud 1997) UN ECE ICP

Continued.

Component	*Performance*	*Performance criterion*	*DLC*	*Degrading agent*	*Estimation of service life*	*Comment*
Concrete Outdoor exposed	Carbonation	Carbonation depth 20 mm	60	Outdoor exposed	Service life more than 100 years	Calculation from (Sarja 1996) Concrete K40
Rendering on facade – STO Vario (figure 4)	Weather protection	Non acceptance of malfunction	60			
Finishing coat Type: Water repellent, organic plaster	Aesthetic, colour changes	Some acceptance of colour changes.	20	Weathering (air pollution, UV-radiation etc.) growth of algae	Washing every 15 to 20 years	Feedback from experience Observations on maintenance intervals
Finishing coat	Protection of render and heat insulation	Non acceptance	30	Weathering	Service life 60 years	Feedback from experience Observations on maintenance intervals Laboratory testing
Rendering	Frost damage	Total degradation in spots	60	Critical moisture contents, freezing temperature.	At least 60 years, dependent of coating	See above
Rendering	Cracking, general	Non visible w=0.2 mm	60	Shrink of underlying material	At least 60 years	See above
Rendering	Cracking in structural joints	Non visible w=0.2 mm	60	Movements of structure	At least 60 years	See above
Fibre-cement board TEPRO Minerit facade board	Aesthetic	Non acceptance of ageing, (cracking, flaking etc.)	30	Weathering (in nordic conditions)	Life expectancy more than 30 years.	Feedback from experience, observations on maintenance intervals. Accelerated ageing tests
Roofing concrete slates (Benders) (figure 3)	Weather protection	Non acceptance of malfunction	60	Weathering (rain, freezing, UV-radiation etc.)	At least 60 years	Feedback from experience Observations on maintenance intervals Laboratory testing

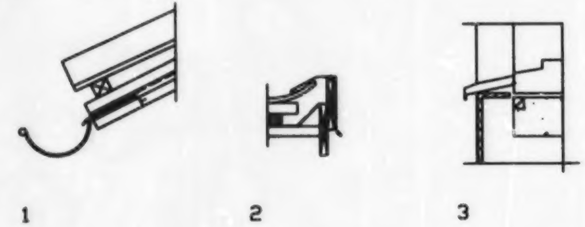


Figure 1. Flashing, coil coated steel, *1: gutters, 2: barge boards, 3: window edge flashing*

Figure 2. The basic build-up of coil-coated sheet metal.

1. Front coating
2. Primer
3. Pre-treatment
4. Zn or Al/Zn (typical 0,6 mm)
5. Steel or aluminium
6. Zn or Al/Zn
7. Pre-treatment
8. Back coating

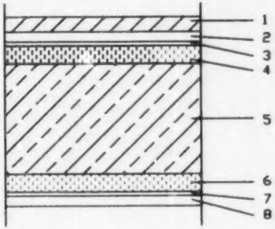


Figure 3.
Roof concrete slates
1 Concrete slate
2 Wood battens
3 Roofing underlay, roofing felt
3 Tongued and grooved board

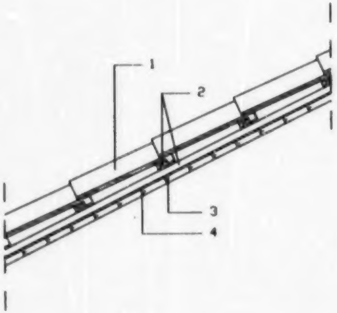
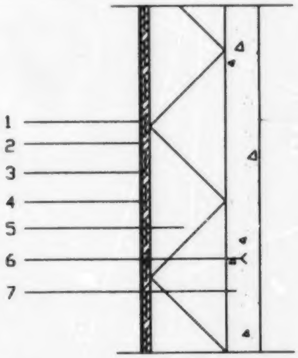


Figure 4.
Exterior wall
1 Finishing coat
2 Primer
3 Rendering
4 Reinforcement
5 EPS heat insulation
6 Mechanical fixing
7 Concrete structure



4 Discussion of service life planning

4.1 Customers demands

In a discussions with representatives of designers, house owners and building management they claim that active service life planning is preferable to today's maintenance programs, which are applied to the building after it has been built.

4.2 Reference service life

A problem is that there are still few tests performed of material and component service life, comprising all the aspects required of the building component when it is in operation in the building, i.e. following the service life prediction methodology [ISO 1999]. A lot of tests are done to explore the degree of frost-resistance, acid-resistance, water repellence, fungi repellence etc. In this work though, the results of such tests have to be considered, as they might be the only available.

The accuracy of the estimated service life is of course suffering from this fact, so one has to discuss if it is worth the effort of doing the estimations or not. If the goal is to find a precise value it is clear that the goal not is reached. But if the goal is to improve the general situation in service life planning the answer is yes.

4.3 Factor method

The factor method in ISO/DIS 15686-1 is meant to be a tool to improve the estimation of the service life. It was found in the project that this method did not improve service life estimations. This opinion is summarised in the following.

Uncertainty of RSLC and values of Factors — The factorial formula (1) comprises in the right side of a reference value (RSLC) and the adjusting Factors, A to G. If the reference value can not be determined accurately it is not appropriate to adjust these values with a set of uncertain Factors.

Uncertainty of the effect by combination of Factors — The method does not support the thoughts that one needs knowledge of cause and effect to estimate the service life. The estimation will be based on uncontrollable occurrences, which can act independently of each other.

5 Conclusions

Service life planning for building components was performed for a whole building. Estimations of service life were based on available data, which have been arranged and evaluated for this purpose. Data were gathered from test-results, interviews, research reports, handbooks. The standard ISO/DIS 15686-1, Buildings: Service Life Planning serves its purpose well by giving a methodology in the work and giving definitions to different items. One observation though is that the Factor method in the standard did not improve the estimations of the service life of a component.

Service life planning is a tool to improve the quality of a building and can be used both in a specific case, as here, and in general estimations of the building stock. All service life estimations should be based on the requirements, degradation environment and performance of a component.

To improve the possibilities to perform an adequate service life planning the following areas of research are suggested:

- Test methods with the aim to predict service life need to be developed.
- In general, computer calculation methods based on the degradation mechanisms need to be developed.
- Service life data need to be collected and be made available to customers through databases, which also should be coupled to design software.

6 Acknowledgements

The work, presented here, is part of a larger project, which also consists of product development and economic measurements. The project is financed with funds from European Union Objective 6, County Administrative Board of Gävleborg, the Swedish Council for Building Research (BFR), the Development Fund of the Swedish Construction Industry (SBUF) and the Concrete Volume Sweden (CVS).

7 References

Caluwerts, P., Sjöström, Ch., Haagenrud S.E., (1996) Service Life Standards: Background and Relation to the European Construction Directive, *Proceedings of 7DBMC*, volume 2, paper 140, ISBN 0 419 22120.

Haagenrud, Svein E, (1997) Environmental Characterisation including Equipment for Monitoring, *CIB W80/RIILEM 140-PSL subgroup 2 Report*, NILU:OR 27/97, Reference O-90043, ISBN 82-425-0875-5.

HAPM, (1992) Component Life Manual, ISBN 0 419 18360 4, London 1992.

Hed, Göran, (1998) Service Life Planning in Building Design, *CIB World Building Congress 1998*, Gävle Sweden 7-12 June, Symposium A, Vol. 1 p. 201-209, ISBN 91-630-6711-0.

Hovde Per Jostein, (1998) Evaluation of the Factor Method to Estimate The Service Life Of Building Components, *CIB World Building Congress 1998*, Symposium A Vol. 1 p. 223-232, ISBN 91-630-6711-0.

ISO/DIS 15686-1 Buildings: Service life planning, Part 1 General principles (1998)

ISO/DIS 15686-2 Buildings: Service life planning, Part 2 Service Life predictions (1999)

Sarja, A, Vesikari, E, (editors) (1996) Rilem report 14, Durability Design of Concrete Structures, ISBN 0 419 21410 0.

Sjöström, Christer, (1990). Service life analysis of organic coatings on sheet metal facade claddings. TRITA-BYMA. Kungliga Tekniska Högskolan, Stockholm 1990:7

Soronis, A.R. Makenya A, (1998) Towards a new integrated ISO - Standard for sustainable building, *CIB World Building Congress 1998*, Symposium A, Vol. 1 p. 467-471, ISBN 91-630-6711-0.

Tolstoy N, Andersson G, Kucera V, Sjöström C, (1989) Utvändiga byggnadsmaterial - mängder och nedbrytning, MOBAK, Statens institut för byggnadsforskning, Gävle, Korrosionsinstitutet, Stockholm, Meddelande M:25, ISBN 91-540-9315-5.

146 SERVICE LIFE PERFORMANCE AUDIT: MEETING CLIENT REQUIREMENTS FOR DURABLE BUILDINGS

G.W. Moss

Building Performance Group Ltd., London UK

Abstract

Construction projects are traditionally costed on the basis of initial capital cost. The privatisation of infrastructure providers, use of private finance for public sector construction projects and ever-tighter constraints on public spending demand the prediction of the overall cost of a project over its whole working life. Until very recently there has been no standard methodology for approaching this problem. ISO/TC59, Building Construction, is developing ISO 15686, a new multi-part International Standard on service life planning for buildings. Part 1 of the standard provides guidance on defining service life requirements during design, and describes two approaches to service life prediction. Part 3 of the standard provides a methodology for evaluating the ability of the design to meet the performance requirements of the client over the whole life of the structure, known as a performance audit. This paper describes the philosophy of the performance audit and illustrates how the concepts have been successfully implemented in the UK by a major public sector client and a latent defects insurer.

Keywords: Audit, durability, performance over time, service life, standards.

1 Introduction

In October 1993 an ISO Working Group (ISO/TC59/SC3/WG9) began work on a new multi-part International Standard on service life planning for buildings. This group has since been elevated to ISO Subcommittee SC14, with separate Working Groups responsible for developing the individual parts of the standard. It is intended that ISO 15686: Buildings - Service Life Planning will be published in five parts as follows:

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- Part 1: General principles
- Part 2: Service life prediction principles
- Part 3: Performance audit
- Part 4: Data requirements, data formatting procedures and data inputs
- Part 5: Life cycle costing and maintenance

A sixth part, on the integration of life cycle assessment techniques in the service life planning of buildings, is also proposed.

At the time of writing this paper, Part 1 was available as a Draft International Standard. The remaining parts were at various stages of drafting. The complete standard is not expected to be available before 2001. The author of this paper is currently employed as drafting consultant to the British Standards committee responsible for drafting Part 3 of the new standard.

A major impetus for the drafting of ISO 15686 has been concern over the construction industry's need to control the cost of building ownership (Cain 1998). As much as 90% of the whole life cost of a building is typically determined during the design stage. Over a 40 year life, maintenance and repair typically account for more than twice the initial capital cost of a building (Flanagan et al 1989). Ever tightening constraints on public spending, the privatisation of infrastructure providers, and the increasing use of private finance for public sector construction projects demand the optimisation of the overall costs of a project over its whole working life. ISO 15686 is intended to address the growing demand for a standard methodology for approaching this problem.

Service life planning aims to reduce the costs of building ownership by optimising the service life performance of the building and the components and materials from which it is constructed. An assessment of how long each part of the building is required to last, helps inform decisions on the most appropriate specification and detailing. When the service lives of the building and its parts are known, maintenance planning, life cycle costing and value engineering techniques can be applied to optimise performance.

Part 1 of ISO 15686 introduces the general principles of service life planning and provides a methodology for defining service life requirements during design. Two approaches to service life prediction, one based on deterioration rates and test data, and one on the factoring of a reference service life, are also described. Further guidance on the testing methodology is provided in Part 2 of the standard. The use of the factor method of service life prediction has been discussed in detail elsewhere (Hovde 1998; Bourke & Davies 1997; Architectural Institute of Japan 1993). This paper is concerned with Part 3 of the standard, 'Performance audit', which provides a methodology for evaluating the ability of the design to meet the performance requirements of the client over the whole life of the building.

2 Overview of ISO 15686-3

The performance audit is an important part of the service life planning of buildings. Parts 1 and 2 of ISO 15686 explain the principles of designing an appropriate service life for different types of building and building component, including how to use test data and experience to assess how long buildings and parts of buildings are likely to last. The performance audit provides a means of ensuring, as far as possible, that the principles and recommendations of Parts 1 and 2 have been put into practice, i.e. that the required service life performance is defined and achieved.

Auditing is a key management tool for ensuring that stated objectives have been met. Comprehensive procedures have been established for auditing quality (ISO 9000 and 10000 series) and for environmental auditing (ISO 14000 series). Many of the principles and techniques used in quality and environmental auditing, including the underlying requirement to check, record and verify, are equally applicable to a service life performance audit.

The performance audit described in Part 3 of ISO 15686 is fundamentally a two stage process, to which additional audit stages may be added if required. The first audit stage focuses on the briefing and outline design stages of a project, to verify that client requirements for performance over time have been identified and clearly stated. The concept of a **design life brief** as a means of communicating these requirements to the design team is introduced, along with auditing procedures for assessing its adequacy as a basis for detailed design. The design life brief identifies the service life performance targets against which the detailed design will later be audited. It should form part of the overall project briefing process.

The second stage of the audit process comprises the **detailed design audit**, which is undertaken once detailed drawings and specifications have been completed, but before the start of construction. The purposes of the audit are:

a) to assess whether the design satisfies the service life performance requirements set out in the design life brief, and;
b) to ensure that adequate information on installation, commissioning and future maintenance is available for those involved in the construction and future operation of the building.

The audit includes an assessment of the service lives and maintenance requirements of key building components and of the adequacy of installation, commissioning and maintenance instructions.

In addition to the two audit stages described above, further audits may be carried out to ensure that the implementation stages of the project (i.e. construction and commissioning) and future maintenance, refurbishment and alteration activities are adequate to achieve the requirements of the design life brief.

The main topics covered in ISO 15685-3 are summarised in Figure 1 below, which is reproduced from the standard.

3 Audit of the design life brief

The design life brief sets out the client's requirements for the service life performance of the building, including any specific component design lives, maintenance requirements or requirements relating to future refurbishment, decommissioning and/or demolition. It is developed during the initial briefing and outline design stages of a project as part of the overall briefing process, and is intended to provide the design team with a clear statement as to the required performance over time of the building. The design life brief should include any specific client requirements for:

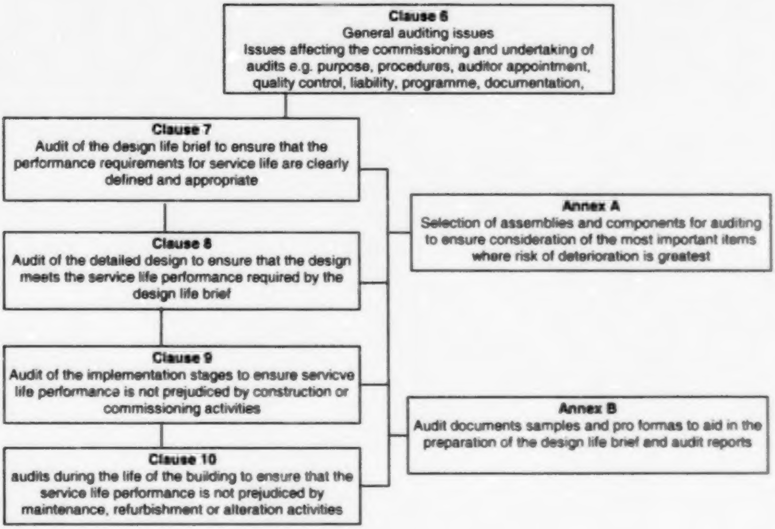
Clause 6
General auditing issues
Issues affecting the commissioning and undertaking of audits e.g. purpose, procedures, auditor appointment, quality control, liability, programme, documentation,
Clause 7
Audit of the design life brief to ensure that the performance requirements for service life are clearly defined and appropriate
Clause 8
Audit of the detailed design to ensure that the design meets the service life performance required by the design life brief
Clause 9
Audit of the implementation stages to ensure servicve life performance is not prejudiced by construction or commissioning activities
Clause 10
audits during the life of the building to ensure that the service life performance is not prejudiced by maintenance, refurbishment or alteration activities
Annex A
Selection of assemblies and components for auditing to ensure consideration of the most important items where risk of deterioration is greatest
Annex B
Audit documents samples and pro formas to aid in the preparation of the design life brief and audit reports

Fig. 1: Overview of ISO 15686-3: Performance audit

- Design life of the building;
- Design lives of specific components, materials and assemblies;
- Reliability of mechanical and electrical plant;
- Future maintenance of the building or specific components;
- Accessibility of specific components;
- Commissioning, maintenance, refurbishment, demolition and disposal.

Information should also be provided on the site conditions, intended use of the building, any specific environmental conditions to be considered (e.g. unusual temperature extremes within the building), procurement method and programme, and the intended management and maintenance regimes for the building.

An audit should be undertaken of the completed design life brief to verify its adequacy as a basis for detailed design. The auditor should report on any deficiencies, such as information that is inadequate, missing or incomplete, contradictory, or unsupported by reliable data. It is then the responsibility of the client and the design team to address any deficiencies in the brief before the detailed design stage commences.

4 Detailed design audit

The detailed design audit is undertaken to assess whether the design conforms to the design life brief, and whether adequate information on installation and commissioning is provided for those involved in the construction phase. The audit alerts the client as to any aspects of the design which do not comply with the stated service life performance requirements, or which may prevent the building or its constituent parts from achieving those requirements. These might include:

- Components, materials or systems whose predicted service life is less than the design life stated in the design life brief;
- Assessed maintenance requirements for the building or its constituent parts that exceed those stated in the design life brief;
- Inadequacies in the installation instructions which might detract from the service life performance of components, materials or systems (eg information which is incomplete, incorrect, unclear or ambiguous).

The audit may also include an evaluation of available information on the future maintenance, refurbishment, adaptation, decommissioning and/or demolition of the building.

As with the audit of the design life brief, any non-conformities or issues requiring further clarification are reported to the client for detailed consideration in conjunction with the design team. Appropriate responses, which might include revisions to the specification and/or installation instructions, provision of additional information, or revision of the design life brief, are fed back to the auditor for further consideration.

5 Further audits

ISO 15686-3 describes a number of additional audits which may be undertaken to ensure that the building is constructed, maintained, repaired, decommissioned, and demolished in accordance with the client's service life performance requirements as set out in the design life brief. These additional audits differ from those described above in that they are concerned with construction activities rather than design development. Typical issues identified during an audit might include:

- Design or specification changes made on site (eg substitution of alternative components or materials);
- Failure to achieve an appropriate standard of workmanship;
- Inadequate storage or protection of components, materials, or work in progress;
- Failure to install components in accordance with manufacturers' instructions;
- Failure to carry out maintenance at the required intervals.

As with the design audits, the objective is to identify any deficiencies in the activities being audited that may detract from the service life performance of the building or its constituent parts.

6 The need for a performance audit

Construction clients are increasingly emphasising their need for reliable predictions of lifetime costs of ownership and for reliable performance over time from their buildings (Cain 1998; Egan 1998; Flanagan 1998). They increasingly regard buildings as factors of production that must meet similar performance targets to their other investments. They point to the widespread application of service life prediction and whole life costing methodology in the development of capital intensive manufacturing and process facilities and seek to apply the discipline to the construction and operation of their facilities.

The service life performance audit is undertaken to provide an assurance that the principles of service life planning described in Part 1 of ISO 15686 have been properly implemented, and, as far as possible, that the building, as designed and constructed, will achieve the required performance over time. The specific objectives of the performance audit will depend upon the requirements of the client (who may be the building owner or occupier, the funding agency, insurer, designer or constructor), and the stage in the project life cycle at which the audit is undertaken.

In the UK it has been found that more than 50% of building failures can be traced back to the brief and to information passed to the contractor in the detailed design documentation. Other failures result from missing or incomplete information, leading to ill-considered decisions on site, and from design changes during the construction stage. Failures can also result from inadequate checks when the building is commissioned, and from inadequate instructions to those responsible for the building's maintenance. The performance audit is intended to identify such deficiencies, wherever possible at the design stage where they can be rectified with minimal cost and disruption.

Auditing procedures similar to those described in this paper are currently in practical use in the UK by a number of organisations including:

- Insurers, to reduce the incidence of defects in social housing schemes;
- Consultants, to optimise the whole life costs of proposed buildings (Moss 1998a);
- Major clients, to verify the service life performance assumptions in contractors' whole life cost models (Cain et al 1998).

7 Case study 1: Social housing latent defects insurance scheme

Auditing procedures similar to those defined in the new standard have been used successfully by UK latent defects insurers for the past eight years, as a risk management tool to reduce the incidence of defects in social housing projects (Holmes & Wornell 1995; Bourke 1996). Housing Association Property Mutual (HAPM) insures new and refurbished housing schemes against latent defects and premature component failure. When the insurance scheme was set up in 1990, actuarial calculations found the cost of insurance to be prohibitively high due to the high historic incidence of defects in social housing developments. A technical auditing process was developed, with the aim of reducing the incidence of defects caused by defective design or construction by at least 50%, thus reducing significantly the insurance premiums.

The design audits are carried out by a team of auditors with the aid of a sophisticated computerised system. The system contains many hundreds of statements, covering the most commonly occurring defects and design deficiencies, grouped according to subject. The auditors inspect the project drawings and specifications and select the most appropriate statement for the issue in question. Exceptions reports are then generated, which list all identified deficiencies or areas requiring further clarification or information. Site audits are undertaken by way of a similar process. Any significant deficiencies that are not adequately addressed by the client and/or design team are likely to be excluded from the insurance cover.

As a result of the auditing process, a significant number of design and construction deficiencies have been identified and rectified, thus preventing premature failures during the life of the buildings. Such deficiencies have included poor detailing between adjacent components, failure to provide adequate fixings or provision for movement, use of incompatible materials, inadequate protection of vulnerable materials, and inappropriate specification of materials. By December 1998, in excess of 85,000 housing units had been audited under the HAPM scheme. Analysis of feedback data from insurance claims has enabled the audit system to be refined and better targeted to actual problem areas. It has also provided guidance to Government Departments on the most commonly occurring design and workmanship deficiencies.

8 Case study 2: UK Ministry of Defence

Service life performance auditing is a key component of the new procurement process currently being developed by the UK Ministry of Defence (Cain et al 1998). Based on the principles of supply chain management which are widely used in petrochemical and aerospace industries, the initiative brings together designers, suppliers, manufacturers, constructors and clients as partners at an early stage in the procurement process in order to optimise on costs and performance. For each project, a whole life cost model is developed and the contractors' service life and maintenance assumptions are subjected to independent audit and validation. Independent third party technical audits are also carried out at key stages in the project to ensure that the design and construction activities meet the client's requirements for performance over time, and to identify any potential causes of building defects or premature component failures.

In the absence of existing published guidance on service life performance auditing, the Ministry of Defence commissioned durability specialists and HAPM technical auditors, Building Performance Group Ltd. to develop a flexible audit methodology for use by first or third party auditors. Published in December 1998, 'Technical audit of building and component durability' (Moss 1998b) provides clear, practical guidance on carrying out technical audits and sets out many of the principles which have since informed the ISO drafting work.

9 Conclusions

This paper has introduced the concept of a service life performance audit, which is the subject of part 3 of a new International standard on the service life planning of buildings. Part 1 of ISO 15686 is concerned with defining service life performance requirements and with assessing the service lives of components and materials in specific situations. Part 3 will provide a means of assessing whether the defined service life performance requirements have been adequately addressed by the design, and whether the building, as constructed, maintained and operated, will perform as required. Auditing is a well-established and accepted management activity, with well-defined procedures for assessing quality and environmental performance. This paper has demonstrated how auditing principles can be applied to the performance over time of buildings and their constituent parts.

It should be noted that Part 3 of ISO 15686 is currently under development and that this paper is based on the first working draft. The drafting consultants would welcome any comments or feedback on the principles set out in this paper, and on the application of service life performance auditing in practice. Interested parties should contact Gary Moss at Building Performance Group Ltd., (email: g.moss@bpg-uk.com).

10 References

Architectural Institute of Japan (1993) *The English edition of principal guide for service life planning of buildings*. Tokyo, Architectural Institute of Japan.

Bourke, K. (1996) Model for insuring the durability of building components. In: Sjostrom, C. (Ed) *Durability of building materials and components 7*. London, E & FN Spon.

Bourke, K. and Davies, H. (1997) *Factors affecting service life predictions of buildings: a discussion paper*. BRE Laboratory report BR 320. Garston, Building Research Establishment.

Cain, C. (1998) Why predict cost of ownership? In: *Whole life costing: the new BS Handbook*. Construction Productivity Network workshop, 28 January 1998.

Cain, C. et al (1998) Building down barriers – Prime contractors' handbook of supply chain management. Sutton Coldfield, Defence Estates Organisation.

Egan, J. (1998) *Rethinking construction: the report of the construction task force*. London, Department of the Environment, Transport and Regions.

Flanagan, R., et al (1998) *A bridge to the future*. Reading Construction Forum report. London, Thomas Telford.

Flanagan, R., et al (1989) *Life cycle costing theory and practice*. Oxford, BSP Professional Books.

Holmes, R. and Wornell, P. (1995) *Premature failure of building components in social housing - managing the risk*. Chartered Institute of Building Construction Papers No.46.

Hovde, P. (1998) Evaluation of the factor method to estimate the service life of building components. In: *Proceedings of CIB World Congress 1998:Materials and Technologies for Sustainable Construction*.

Moss, G. (1998a) *OCAP: Occupancy cost appraisal and profiling*. London, Building Performance Group Ltd.

Moss, G. (1998b) *Technical audit of building and component durability*. London, Building Performance Group Ltd.

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